TA705.3G8L682002c.1

GEOTECHNICAL ENGINEERING REPORT

LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT TASK 2.4.4 – UPRR TO ROUTE 237 SANTA CLARA COUNTY, CALIFORNIA

Prepared for
CH2MHILL
1737 North First Street, Suite 300
San Jose, CA 95112

November 8, 2002

URS

55 South Market Street, Suite 1650
San Jose, California 95113

TA
705.3
G8.L68
2002
c.1
LIBRARY

Project 95-00112030.00 / 28648808

URS

November 8, 2002
Project 95-00112030.00 / 28648808

Mr. David Von Rueden
CH2MHILL
1737 North First Street, Suite 300
San Jose, CA 95112

Subject: Geotechnical Engineering Report
Lower Guadalupe River Flood Control Project
Task 2.4.4 UPRR to Route 237
Santa Clara County, California

Dear Mr. Von Rueden:

In accordance with our Purchase Order/Subcontract 51347 dated August 14, 2001, we are pleased to submit this revised geotechnical engineering report for the Lower Guadalupe River Flood Control Project, to be constructed in Santa Clara County, California. This report addresses the portion of the project alignment between the Union Pacific Railroad (UPRR) and Route 237, and is supplemental to our previous engineering report entitled "Geotechnical Investigation Lower Guadalupe River Flood Control Project," dated April 28, 2000.

The accompanying report presents our opinions and recommendations regarding the geotechnical aspects influencing design and construction of the proposed levee improvements. The opinions and recommendations are based upon review of available information, the results of field investigation, laboratory testing, engineering analyses we conducted as part of our scope, as well as engineering judgment and local experience.

Mr. Thomas Pennington, Staff Engineer, and Ms. Jennifer Werle, Staff Engineer, assisted in the selection of the engineering parameters, the engineering analysis of levee stability and potential levee seepage, as well as preparation of the report. Mr. Pennington also analyzed potential settlement of the levees. Mr. Michael L. Larson, Principal Geotechnical Engineer, provided peer review for all geotechnical aspects of the project. This peer review was in accordance with the URS Corporation Quality Control Plan.

Since our report of July 19, 2002 was issued, we received review comments from CH2MHILL design team members on September 5, 2002 and met with the team members on September 18, 2002 to discuss the comments. We also received comments from the Santa Clara Valley Water District staff on October 21, 2002. On September 17, 2002, we received a set of plans stamped "Preliminary, Current as of September 17, 2002." Subsequently, we have incorporated all review comments and updated this report to include the geotechnical design parameters consistent with the latest project data we have received.

URS Corporation
55 South Market Street, Suite 1650
San Jose, CA 95113
Tel: 408.297.9585
Fax: 408.297.6962

We are pleased to have been of service to you on this project. If any questions should arise, please contact our office.

Sincerely,

Stephen Huang, G.E. 02150
Deputy Project Manager

Paul J. Boddie, G.E. 00152
Project Manager

Table of Contents

SECTION 1 INTRODUCTION 1-1

1.1 Project Description 1-1

1.2 Scope of Work 1-5

1.3 Previous Investigations 1-6

1.4 Data Review 1-7

SECTION 2 SITE GEOLOGY AND SEISMICITY 2-1

2.1 Geologic Setting 2-1

2.2 Regional Faults And Seismic Sources 2-1

2.2.1 San Andreas Fault 2-1
2.2.2 Hayward Fault 2-2
2.2.3 Hayward Fault - Southeast Extension 2-2
2.2.4 Calaveras Fault 2-3
2.2.5 Monte Vista - Shannon Fault 2-3
2.2.6 Sargent - Berrocal Fault 2-3

2.3 Ground Motions 2-4

2.4 Summary 2-4

SECTION 3 SITE AND SOIL CONDITIONS 3-1

3.1 Surface Conditions 3-1

3.1.1 Field Reconnaissance 3-1
3.1.2 General 3-1
3.1.3 Segment From UPRR Tracks To Route 237-Station 7+000 To Station 8+000 3-1
3.1.3.1 East Levee 3-2
3.1.3.2 West Levee 3-2
3.1.3.3 Mitigation Levee 3-2

3.2 Subsurface Conditions 3-3

3.2.1 Field Exploration 3-3
3.2.2 Laboratory Testing 3-3
3.2.3 Soil and Groundwater Conditions 3-3
3.2.3.1 East Levee 3-4
3.2.3.2 West Levee 3-5
3.2.3.3 Mitigation Levee 3-5
3.2.3.4 River Channel 3-6

SECTION 4 LEVEE SLOPE STABILITY EVALUATION 4-1

4.1 Cross Sections Selected For Analysis 4-1

4.1.1 Selection of 23 Levee Cross Sections 4-1
4.1.2 Selection of Six Levee Cross Sections for Task 2.4.4 4-1
4.1.3 Selection of Shear Strength Parameters 4-2

4.1.3.1 Cohesive Soils in Levee and Levee Foundation....4-2
4.1.3.2 Cohesive Soil in Channel....4-4
4.1.3.3 Granular Soils....4-5
4.2 Method of Analysis....4-5
4.2.1 Pore Pressures....4-6
4.2.2 Static Loading Conditions....4-6
4.2.2.1 End of Construction....4-7
4.2.2.2 Sudden Drawdown....4-7
4.2.2.3 Steady Seepage From Full Flood Stage....4-7
4.2.3 Earthquake Loading Conditions....4-7
4.3 Stability Results of Levee Sections....4-8
4.4 Stability Results of Depressed Maintenance Roads....4-9
SECTION 5 SETTLEMENT EVALUATION....5-1
5.1 Subsidence....5-1
5.2 Levee Settlement....5-1
5.2.1 Inferred Settlement....5-1
5.2.2 Predicted Settlement....5-1
5.3 Future Settlement....5-3
SECTION 6 SEEPAGE EVALUATION....6-1
6.1 Method of Analysis....6-1
6.2 Cross Sections And Results of Analysis....6-1
SECTION 7 FLOODWALL, WINGWALL AND MSE WALL DESIGN PARAMETERS 7-1
7.1 Background....7-1
7.2 Recommendations for Floodwalls....7-1
7.2.1 Corps Requirements....7-1
7.2.2 Wall Settlement....7-2
7.2.3 Driving Side Design Earth Pressure....7-4
7.2.4 Resisting Side Design Earth Pressure....7-4
7.2.4.1 At Rest Earth Pressure....7-4
7.2.4.2 Passive Pressure....7-5
7.2.5 Bearing Capacity....7-5
7.2.6 Foundation Uplift....7-5
7.2.7 Drilled Piers....7-6
7.2.8 Lateral Load Capacity of Drilled Piers....7-6
7.2.9 Construction Considerations for Drilled Piers....7-7
7.3. Recommendations for Wingwalls....7-7
7.3.1 General....7-7
7.3.2 Lateral Earth Pressure....7-7
7.3.3 Deep Foundations....7-8

7.4 MSE Wall Design 7-8
7.4.1 Wall Configuration 7-8
7.4.2 Materials 7-8
7.4.3 Seepage Considerations 7-9
7.4.4 Driving Side Lateral Pressures 7-10
7.4.5 MSE Stability 7-10
7.4.5.1 Sliding and Overturning Stability 7-10
7.4.5.2 Global Stability 7-11
7.4.6 Bearing Capacity 7-11
7.4.7 Wall Settlement 7-12
7.4.8 Leveling Pad 7-12
7.4.9 Earthquake Considerations 7-12
7.5 Corrosion Investigation 7-13

SECTION 8 LIQUEFACTION POTENTIAL 8-1

SECTION 9 SITE PREPARATION AND EARTHWORK 9-1

9.1 General 9-1
9.2 Clearing and Demolition 9-1
9.3 Grubbing 9-1
9.4 Stripping 9-1
9.5 Removal of Sediment in Channel 9-2
9.6 Excavation 9-2
9.7 Subgrade Preparation 9-2
9.8 Fill Materials 9-3
9.8.1 Approval 9-3
9.8.2 Onsite Materials 9-4
9.8.2.1 Channel Sediment 9-4
9.8.2.2 Excavation for Floodwalls, MSE Walls, Wingwalls and Tieback Walls 9-4
9.8.2.3 Geotextile Soil Reinforcing Walls 9-4
9.8.2.4 Existing Armoring Aggregate 9-4
9.8.3 Import Materials 9-4
9.8.3.1 General Levee Fill 9-4
9.8.3.2 Bridging Material 9-5
9.8.3.3 Armoring Aggregate 9-5
9.9 Fill Placement and Compaction 9-5
9.10 Construction Dewatering 9-6

SECTION 10 CONCLUSIONS AND RECOMMENDATIONS 10-1

10.1 General 10-1
10.2 Results of Slope Stability Evaluation 10-1
10.2.1 Levee Slopes 10-1

10.2.2 Depressed Maintenance Road Slopes 10-1
10.3 Results of Settlement Analyses 10-1
10.4 Results of Seepage Analyses 10-2
10.5 Site Preparation and Grading 10-3
10.6 Liquefaction Potential 10-3
SECTION 11 LIMITATIONS 11-1
SECTION 12 REFERENCES 12-1

TABLES

Table 1	Existing Boring Log Index List
Table 2a-2d	Estimated 84th percentile Peak Horizontal Accelerations in g's at Segments 1 through 4
Table 3	Subsurface Characterization
Table 4	Proposed 6 Cross Sections of Levees Considered to be Most Critical
Table 5	Summary of Effective Shear Strength Parameters Based on Isotropically Consolidated Undrained Triaxial Tests
Table 6	Army Corps of Engineers Slope Stability Design Requirements

FIGURES

Figure 1	Site Map
Figure 2	Site and Field Exploration Location Plan
Figure 3	One Percent Chance Flood Hydrograph
Figure 4	Regional Active Faults
Figure 5	Stability Analysis, Cross Section 8 (Station 7+170 East Levee)
Figure 6	Stability Analysis, Cross Section 9 (Station 7+170 West Levee)
Figure 7	Stability Analysis, Cross Section 10 (Station 7+320 West Levee)
Figure 8	Stability Analysis, Cross Section 11(Station 7+800 East Levee)
Figure 9	Stability Analysis, Cross Section 12 (Station 7+950 West Levee)
Figure 10	Stability Analysis, Cross Section 13 (Station 7+997 East Levee)
Figure 11	Index Property Plots for Cross Section 8
Figure 12	Index Property Plots for Cross Section 9
Figure 13	Index Property Plots for Cross Section 10
Figure 14	Index Property Plots for Cross Section 11
Figure 15	Index Property Plots for Cross Section 12
Figure 16	Index Property Plots for Cross Section 13
Figure 17	Embankment Strength Reduction Factor for Soft Clay Foundations
Figure 18	Shear Strength Parameters for Lean Clay Fill with Dry Density less than 110 pcf
Figure 19	Shear Strength Parameters for Lean Clay Fill with Dry Density greater than 110 pcf
Figure 20	Shear Strength Parameters for Fat Clay with Dry Density less than 80 pcf
Figure 21	Shear Strength Parameters for Lean Clay with Dry Density between 80 pcf and 100 pcf
Figure 22	Shear Strength Parameters for Lean Clay with Dry Density between 100 pcf and 110 pcf
Figure 23	Relationship between Standard Penetration Resistance, Relative Density and Effective Overburden Pressure
Figure 24	Correlations of Strength Characteristics for Granular Soils
Figure 25	Stability Analysis, Depressed Maintenance Road, Station 7+200 West Levee

Figure 26	Stability Analysis, Depressed Maintenance Road, Station 7+200 West Levee
Figure 27	Stability Analysis, Depressed Maintenance Road, Station 7+200 West Levee
Figure 28	Stability Analysis, Depressed Maintenance Road, Station 7+200 East Levee
Figure 29	Stability Analysis, Depressed Maintenance Road, Station 7+200 East Levee
Figure 30	Stability Analysis, Depressed Maintenance Road, Station 7+200 East Levee
Figure 31	Stability Analysis, Depressed Maintenance Road, Station 7+260 East Levee
Figure 32	Time Rate of Settlement for Cross Section 8
Figure 33	Time Rate of Settlement for Cross Section 9
Figure 34	Time Rate of Settlement for Cross Section 10
Figure 35	Time Rate of Settlement for Cross Section 11
Figure 36	Time Rate of Settlement for Cross Section 12
Figure 37	Time Rate of Settlement for Cross Section 13
Figure 38	Permeability and Drainage Characteristics of Soils
Figure 39	Steady State Seepage Analysis, Cross Section 8
Figure 40	Steady State Seepage Analysis, Cross Section 9
Figure 41	Steady State Seepage Analysis, Cross Section 10
Figure 42	Steady State Seepage Analysis, Cross Section 11
Figure 43	Steady State Seepage Analysis, Cross Section 12
Figure 44	Steady State Seepage Analysis, Cross Section 13
Figure 45	Time Dependent Phreatic Surface Lines, Cross Section 8
Figure 46	Time Dependent Phreatic Surface Lines, Cross Section 9
Figure 47	Time Dependent Phreatic Surface Lines, Cross Section 10
Figure 48	Time Dependent Phreatic Surface Lines, Cross Section 11
Figure 49	Time Dependent Phreatic Surface Lines, Cross Section 12
Figure 50	Time Dependent Phreatic Surface Lines, Cross Section 13
Figure 51	Typical MSE Wall Configuration With Force Diagram

APPENDICES

Appendix A	1997 Guadalupe Mitigation Site Drawings and 1997 Geotechnical Investigation Report
Appendix B	Field Investigation: Exploratory Borings
Appendix C	Laboratory Testing
Appendix D	Preliminary Geotechnical Design Floodwall Parameter Letters
Appendix E	Corrosion Investigation

SECTION 1

This report presents the results of a geotechnical investigation for the Lower Guadalupe River Flood Control project for Levee Improvements from the Union Pacific Railroad (UPRR) to State Route 237 (SR 237). The location and layout of the project are shown in Figure 1, Site Map. A more detailed layout of the project, including the approximate centerline of the existing West Levee and East Levee, is shown in Figure 2, Site and Field Exploration Location Plan. This figure includes the design control line with stationing presented in meters, as well as estimated limits of historical meanders and locations of existing and new field exploration points. Locations of streets and highways crossing the Guadalupe River are also shown in Figure 2. Topographic maps with aerial photography background were presented as part of our April 28, 2000 report for this project. Throughout the text of this report, elevation, distance and stationing will be presented in metric units; except for stationing, English units will also be presented in parenthesis. According to the latest design drawings with sheets dated June 28, July 1 and July 3, 2002 (referred to as July 2002 Design Drawings from this point onwards) there are three separate stationing systems, including the "P" Line, "E" Line and "W" Line. The "P" Line traverses the approximate centerline of the channel; the station numbers presented in this report are based on the "P" Line unless stated otherwise. The "E" Line traverses the crest of the East Levee, i.e. looking upstream. The "W" Line traverses the crest of the West Levee, i.e. looking upstream. All graphs, charts and laboratory test results are presented in English units.

The report presents the results of (1) a review of the available subsurface information, construction records and as-built drawings, (2) the subsurface exploration and laboratory testing program conducted for this study, and (3) the results of engineering evaluations that serve as the basis for our opinions and recommendations regarding the geotechnical engineering aspects of the project.

The goal of this study is to provide a design level geotechnical investigation for the Lower Guadalupe River in order to bring the levees into compliance with the requirements of the Federal Emergency Management Agency (FEMA) and the United States Army Corps of Engineers (Corps). Ultimately, the Santa Clara Valley Water District (District) wishes to obtain FEMA certification of the levees in order to receive a Letter of Map Revision (LOMR).

To meet FEMA standards, the levees must provide a minimum freeboard of 0.9 meter (3 feet) above the 100-year (1-percent) flood. An additional 0.3 meter (1 foot) above the minimum is required within 30.5 meters (100 feet) on either side of structures (such as bridges) riverward of the levees or wherever the flow is constricted. An additional 152 mm (1/2 foot) above the minimum at the upstream end of the levee, tapering to not less than the minimum at the downstream end of the levee, is also required. The District also must demonstrate that the levees are stable from a geotechnical standpoint.

1.1 PROJECT DESCRIPTION

The Lower Guadalupe River has been defined as that portion of the river that extends from I-880 to the river's terminus with the San Francisco Bay at Alviso. This portion of the Guadalupe River is located in the City of San Jose and Santa Clara County,

California. This report addresses the portion of the levee alignment between the UPRR (Station 7+008) and Route 237 (Station 8+000). Consequently, the levee section analyzed for this study extends approximately 992 meters (3,254 feet). The District currently proposes to meet the freeboard requirements with the construction of floodwalls within this reach. It also includes West Levee realignment immediately north of Route 237 and outboard of the existing West Levee. This area is defined as the "Mitigation Levee" for the Santa Clara County Transit District. The location of the Mitigation Levee is shown in Figure 2; it was constructed in 1999.

According to Sheets S-1 through S-19 of the July 2002 Design Drawings, floodwalls are planned with the lengths and heights as described in the following table.

Floodwall	Station	Location	Approximate Length (m)	Wall Height (mm)		Sheet Numbers	Maximum Fill Height (m)
				Minimum	Maximum		
FW2E	E20+990.992 To E21+073.575	East levee, east of UPRR	86.418	1,200	1,800	S-3	0.9
FW3E	E21+105.013 To E21+509.563	East levee, east of Gold Street bridge	413.700	1,200	2,100	S-4 to S-6	1.2
FW4E	E21+530.069 To E21+954.775	East levee, north of State Route 237 bridge	422.300	1,200	2,100	S-7 to S-9	1.2
FW1W	W40+293.340 To W40+391.385	West levee, east of UPRR	113.474	1,200	2,700	S-14	1.5
FW2W	W40+403.195 To W40+491.549	West levee, east of Gold Street bridge	91.86	1,500	2,100	S-15	1.3
FW3W	W40+524.256 To W40+831.571	West levee, east of FW2W, west of Mitigation Levee	308.039	1,200	2,100	S-16 to S-18	1.2
FW5W	W40+842.291 To W40+899.000	Mitigation Levee	77.239	1,800	2,400	S-19	1.4

Based on the above table, the total length of floodwalls is about 1,513 meters (4,964 feet). Wall height H ranges from 1.2 to 2.7 meters (3.9 to 8.9 feet); H is defined from top of floodwall to top of footing. Since the minimum burial depth is 0.75 meter (2.46 feet), the wall height above finished grade ranges from 0.45 to 1.95 meters (1.48 to 6.40 feet). Furthermore, fill height ranging from 0.9 to 1.5 meters (3.0 to 4.9 feet) is planned behind six (6) floodwalls FW2E, FW3E, FW1W, FW2W, FW3W and FW5W.

According to a mark-up plan received on May 24, 2002 and the July 2002 Design Drawings, Mechanically Stabilized Embankment (MSE) walls are planned with the lengths and heights described in the following table.

MSE Wall	Station	Length (m)	Height Range (H, m)	Strip Length = 1.2*H (m)	Sloping Backfill?
MSW1E	E21+470.100 To E21+569.671	100.635	2.4-3.0	2.9-3.6	No
MSW5W	W40+464.600 To W40+559.216	94.650	1.8-2.4	2.2-2.9	No

Based on the above table, the total length of MSE walls is about 195 meters (641 feet). Wall height H ranges from 1.8 to 3.0 meters (5.9 to 9.8 feet). Except for one wing wall with a backfill slope of 2 to 1 (horizontal to vertical), most of the walls will have a relatively level backfill surface. Within Reach G the levee crown will be raised a maximum height of about 1.2 meters (3.9 feet) at the maintenance road ramps.

According to sheets CD-11 and CD-12 of the July 2002 Design Drawings, wingwalls for the Gold Street bridge are planned at Abutment 1 (south abutment) and Abutment 6 (north abutment), respectively. Two wingwalls are shown at each abutment. Wall lengths range from 4.500 to 7.450 meters (14.8 to 24.4 feet) and wall heights vary from 4.285 to 4.525 meters (14.0 to 14.8 feet). Bottom of pile cap at all four wingwalls is positioned at Elevation 2.000 meters (6.6 feet).

CH2MHILL has designated seven (7) reaches for the entire Lower Guadalupe River Flood Protection Project as described in the following table.

Reach	Station Limits		Description	
	From	To	From	To
G	7+000	8+000	Downstream of UPRR bridge	Downstream of Route 237
F	8+000	9+200	Downstream side of Route 237	Downstream of Tasman Dr.
E	9+200	10+100	Downstream of Tasman Dr.	Downstream of Hetch Hetchy Aqueduct
D	10+100	11+800	Downstream of Hetch Hetchy Aqueduct	Downstream of Mont. Expwy
C	11+800	13+650	Downstream of Mont. Expwy	Downstream of Trimble Road
B	13+650	14+500	Downstream of Trimble Rd	Downstream of Highway 101
A	14+500	15+510	Downstream of Highway 101	Station 15+510

As this report is for Task 2.4.4 only, the geotechnical engineering analysis, conclusions and recommendations are limited to Reach G only.

According to District As-built drawings (1995), portions of the existing West and East levees were raised above their existing crest by (1) placement of "impervious sandy clay, PI less than 25±5" or (2) construction of L-shaped reinforced concrete floodwalls, or (3) placement of engineered fill within layers of geotextile soil reinforcing (GSR). This resulted in steepening of the slopes of the raised portions generally to 1.75 to 1

(horizontal to vertical) on the riverside; whereas slopes on the landside were generally 1.5 to 1 for clay fill and 1 to 1 or 1.75 to 1 for areas with GSR. For GSR levee improvements, 2 or 3 horizontal fabric layers were installed and wrapped around on the outside (landside) face. The 1995 drawings indicate that "semi-impervious, or structural backfill, PI less than 15±5" was placed inside the outermost 1.5 meters to 2.7 meters (5 to 9 feet) landside portion of the GSR. The following table describes the locations where GSR was used between Gold Street and I-880.

Levee	Limits of Geotextile Soil Reinforcing (GSR)			Length		Wall Height		Comments
	Begin (feet)	End (feet)	Location	(feet)	(meters)	(feet)	(meters)	
East	1098+44	1111+30	South of Tasman Drive	1,286	392.0	2.5-3.83	0.76-1.17	5 to 9 feet of "semi-impervious" or "structural" backfill"
East	1092+30	1096+27	North of Tasman Drive	397	121.0	2.5-3.83	0.76-1.17	5 to 9 feet of "semi-impervious" or "structural" backfill"
West	84+90	97+94	North of Tasman Drive	1,304	397.6	2.5-3.83	0.76-1.17	5 to 9 feet of "semi-impervious" or "structural" backfill"
East	1052+00	1058+25	North of Route 237	625	190.6	2±	0.61±	Assumed 5 feet of "semi-impervious" or "structural backfill"
West	45+70	52+50	Between Gold Street and Route 237	680	207.3	2±	0.61±	Assumed 5 feet of "semi-impervious" or "structural backfill"

More specifically, the total length of GSR wall for Task 2.4.4 is 397.9 meters (1,305 feet) and for Task 2.4.5 is 910.6 meters (2,988 feet).

As in the previous study (URS, April 2000), we have used the NGVD survey datum for the elevation of the ground surface at all explorations and subsurface profiles; both NGVD English and Metric units are provided in this report text. In the event that conversion to NAVD is desired in metric units, the following equation should be used:

$$\text{NAVD (metric)} = \text{NGVD (metric)} + 0.81$$

The elevations shown in the August 2001 and December 2001 drawings are based on the NAVD datum.

The 100-year (1-percent) flood hydrograph provided by the District and developed by the Corps is shown in Figure 3. As shown, the peak discharge is 481.4 cubic meters per second or cms (17,000 cubic feet per second or cfs); the duration of flow exceeding 453 cms (16,000 cfs) only lasts about 12 hours. When the discharge rate is reduced to 113 cms (4,000 cfs) or about one-quarter of the peak discharge, the time interval is about 42 hours.

1.2 SCOPE OF WORK

The following scope of work has been performed to serve as a basis for the development of the opinions and recommendations presented in this report:

- Reviewed available information including the following:
  - Current levee cross sections, topographic drawings, and design drawings provided by the District and CH2MHILL.
  - Plans and previous borings for the Mitigation Levee
- Selected locations of new borings.
- Cleared utilities at field exploration points through Underground Service Alert.
- Performed field exploration including:
  - 7 borings to depths of 10.5 to 13.6 m (34.5 to 44.5 feet); boreholes were grouted upon completion.
  - Includes Borings EB-18, EB-19, EB-20, WB-18, WB-19, WB-20 and CB-01.
- Performed laboratory testing, including:
  - Grain size distribution
  - Plasticity index
  - Unconfined compression, water content and dry density
- Performed preliminary geotechnical assessment.
- Identified key geotechnical cross-sections along both levees.
- Characterized soil profiles for these cross-sections.
- Identified potential geotechnical issues.
- Selected six (6) cross-sections judged to be most critical.
- Performed geotechnical calculations:
  - At six levee cross sections, 3 East Levee and 3 West Levee
  - Factors of safety for slope stability for 4 loading conditions
  - Slope stability earthquake deformations
  - Settlement
  - Seepage
  - In accordance with Corps Engineer Manual No. EM 1110-2-1913 (Design and Construction of Levees) and Corps Engineer Manual No. EM 1110-2-1904 (Soil Mechanics Design - Settlement Analysis)
  - At three depressed road locations, estimate factors of safety for slope stability for end-of-construction condition only
  - Floodwall design parameters
  - Wingwall design parameters
  - MSE wall design parameters (also included global stability and seepage evaluation)

- Prepared revised geotechnical report:
  - Summarized the results of all the above tasks
  - Developed geotechnical design parameters for the reinforced concrete floodwalls
  - Developed geotechnical design parameters for the reinforced concrete wingwalls
  - Developed geotechnical design parameters for the MSE walls
  - Developed geotechnical opinions and recommendations regarding the structural integrity of both levees.
  - Developed recommendations for site preparation and earthwork

1.3 PREVIOUS INVESTIGATIONS

We have previously performed an investigation for the subject project. The results of that investigation were presented in a report entitled "Geotechnical Investigation, Lower Guadalupe River Flood Control Project, Santa Clara County, California," dated April 28, 2000. That report included the entire alignment of the project from the Union Pacific Railroad (UPRR) to I-880. We performed slope stability, seepage, and settlement analyses, and provided recommendations based on the selection of seven critical cross sections. One cross section (Cross Section 1 as shown on Figure 2) from the 2000 study is within the limits of the current study.

The logs of 185 previous borings completed along the levees or for projects in the immediate vicinity of the levees are provided in Appendix A of the 2000 report. The logs represent a total of 26 different projects, over the period of 1960 through 1997. The index list for each project is shown in Table 1; a different boring number index is assigned for each project. The locations of the previous borings and CPTs (along with the new borings) are shown in Figure 2, Site and Field Exploration Location Plan. The depth of these borings ranges from 2.4 to 30.8 meters (8 to 101 feet). Most of the existing boring logs are from investigations previously performed by URS' predecessor firm, Woodward-Clyde Consultants. The existing borings shown on Figure 2 were selected based on these two criteria:

- They were located within or in close proximity to the footprint of the levees; and
- The technical information provided on the log was reliable, in our engineering judgment.

It should be noted that borings in the initial investigations in the early 1960s were drilled prior to construction of the levees and therefore reveal subsurface conditions only in the foundation. Subsequent borings drilled mostly in the 1980s and 1990s extended through the levee section and therefore revealed the levee fill and underlying foundation conditions at that time.

Most of the routine laboratory test data are shown on the individual boring logs, i.e. water content, dry density, and unconfined compression. The results of more complex laboratory tests such as grain size distribution, plasticity index, consolidation and consolidated undrained triaxial compression (with pore pressure measurements) from the previous investigations are presented in Appendix B of the 2000 report.

1.4 DATA REVIEW

As part of the April 2000 investigation, we reviewed the available levee design and construction information, as well as related records, mostly provided by the District including the following:

- Drawings for the original 1963 river relocation
  - 16 sheets (some as-built) dated 1959 and 1960 and entitled "Map and General Construction Plans of Zone C-1 (Central) Project No. 3, Guadalupe River Improvement, San Francisco Bay to Branham Lane, Santa Clara County Flood Control and Water Conservation District, Santa Clara County California"; most drawings prepared by Creegan and D'Angelo.
  - 54 sheets (some as-built) dated April 26, 1963 and entitled "Map and General Construction Plans of Guadalupe River Unit 1 From Montague Road to Alviso Slough, Zone C-1 (Central), Project No. 30016, Santa Clara County Flood Control and Water Conservation District, Santa Clara County, California"; most drawings prepared by Creegan and D'Angelo.
- As-built drawings for the 1983 levee raising including 116 sheets dated September 27, 1984(as-built) and entitled "Map and Construction Plan for Guadalupe River From S.P.R.R. to Highway 101, Central Flood Control Zone, Project No. 3015, Santa Clara Valley Water District"; logs of 10 soil borings were included on 3 plan sheets.
- As-built drawings for the 1995 interim raising between UPRR and Highway 101, including 15 sheets dated April 11, 1996 (as-built) entitled "Map and General Plan for Guadalupe River From Southern Pacific Railroad to Highway 101, Central Flood Control Zone, Project No. 3015, Santa Clara Valley Water District".
- Approximately 240 current levee cross sections.
- Current topographic and aerial photography drawings including 15 sheets dated November 25, 1997; no title; from U.P.R.R. bridge to south of Highway 101 (Airport Island Bridge). Contours are plotted on both levees using a 1.0 meter contour interval.
- Existing elevations of crest of both East and West Levees. Settlement was estimated by subtracting as-built levee crest elevations from the existing elevations.
- A graphical plot of land subsidence from a United States Geological Survey paper entitled "Land Subsidence in the Santa Clara Valley, Alameda, San Mateo and Santa Clara Counties, California", 1971, by J.F. Poland.

We have also reviewed available District drawings and other historical records for locations of meanders. As shown on Figure 2 Site and Field Exploration Location Plan, the approximate channel limits of old meanders are plotted for almost the entire length of the project; these are based on District drawings dated 1960 and 1963. Between Tasman Drive and Montague Expressway, the locations of earlier meanders are based on a 1873 map obtained from the archives of the San Jose Historical Museum ("Map of a Re=Survey of the "Lick" Property, Party, Part of "Rancho Rincon De Las Esteros",

surveyed October 31, 1873 by J.H. Pieper and Re=Surveyed July 1887 by H.A. Braissard) were identified and plotted on Figure 2.

During construction of the levees, we understand the District typically required the removal of soft deposits within the meander and within the footprint of the levee and replacement with soil spread in layers and compacted by track walking with a bulldozer or other track-mounted equipment. However, no specific relative compaction requirements were set.

As part of the current investigation, we reviewed the available levee improvement design information, mostly provided by CH2MHILL, including the following:

- "Santa Clara Valley Water District, Lower Guadalupe River Planning Study, Engineer's Report, Project No. 3015" by CH2MHILL, Inc. dated August 2001.
- Drawings for the proposed levee improvements in the plans entitled "Map and General Plan for Lower Guadalupe River Flood Control Project from I-880 to Union Pacific Railroad (UPRR) Bridge", Project No. 3015, dated August 2001.
- "Santa Clara Valley Transportation Authority, Tasman Corridor Mitigation and Monitoring Plan, USACE File # 18881S92, Project No. 725-03" by H.T Harvey & Associates dated January 31, 1997. Appendix G of their report contains a report entitled "Geotechnical Investigation Report, Proposed Levee and Improvements, Guadalupe River Site, San Jose, California", by Geo/Resource Consultants, Inc. dated January 10, 1997 (Project Number 1935-000). Their field exploration program included two test pits (Test Pits A and B to maximum depth of 8.0 feet) and six borings (Borings B-1 through B-6) to depths of 25 to 35 feet. A copy of the field exploration plan, logs of borings, test pits and laboratory test results are included in Appendix A of this report. For their geotechnical analyses, the proposed levee had side slopes of 2 horizontal to 1 vertical, crest at Elevation 6.3 m (18.0 feet plus 2.66 feet which equals 20.66 feet) and crest width of 4.9 m (16 feet). They performed slope stability analyses for four loading conditions. In addition, they performed a settlement analysis of the levee. They estimated long term total settlement "ranging from 9 inches to a maximum of 3.6 feet. The majority of the settlement will occur within 25 years after construction....In most of the levee alignment, however, we anticipate that the total settlement to be on the order of 1 foot."
- Preliminary drawings for the Mitigation Levee entitled "Tasman Corridor Project, Guadalupe Mitigation Site" dated January 16, 1997 by ANWest, Inc. for the Santa Clara Valley Transportation Authority; included are nine sheets: G-1 and civil sheets C-1 through C-8. These are included in Appendix A. These drawings indicate this levee has side slopes of 2:1 (horizontal to vertical), a crest width of 5.5 m (18 feet) and a crest finished grade at generally Elevation 6.3 m (18.0 feet).
- "Supplemental Design Memorandum, 30-Percent Design Submittal, Lower Guadalupe River Flood Protection Project, Alviso Marina to Interstate 880, Project No. 3301511" by CH2MHILL, dated December 7, 2001.

- Drawings for the proposed levee improvements in the plans entitled "Map and Construction Plan for Lower Guadalupe River Flood Protection Project from the Alviso Marina to Interstate 880", Project No. 301511, dated December 7, 2001.

- Drawings for the proposed levee improvements in the plans entitled "Map and Construction Plan for the Lower Guadalupe River Flood Protection Project from the Alviso Marina to Interstate 880", Project No. 301511, dated April 29, 2002.

SECTION 2

This section was included in the April 2000 report, and is repeated herein for ease of reference. No additional study was performed as part of the current investigation, except Section 2.3 "Ground Motions" has been modified.

2.1 GEOLOGIC SETTING

The levees span from the northerly end of the Santa Clara Valley in Alviso to nearly the heart of the Valley in San Jose; the project area lies between the Santa Cruz Mountains to the west and the Diablo Range to the east. The Santa Clara Valley is a broad northwesterly trending alluvial filled basin. The older Mesozoic rocks of the Santa Cruz Mountains and the Diablo range underlie the thick accumulations of alluvial sediments in the Santa Clara Valley. These alluvial sediments are divided into the lower older Santa Clara Formation, and the upper younger surficial deposits of alluvium, and alluvial fans.

Depth to bedrock is estimated to be at least 92 meters or 300 feet (Rogers and Williams, 1974) along the levee alignment.

2.2 REGIONAL FAULTS AND SEISMIC SOURCES

The active faults that could potentially generate earthquakes producing strong ground motions along the Guadalupe River Flood Control Improvement Project are described below and are shown on Figure 4, Regional Active Faults. Each fault is characterized in terms of the type of faulting, rupture length, dip, maximum magnitude, and slip rate. The 84th percentile peak horizontal accelerations potentially generated by the maximum earthquake for each seismic source are tabulated in Tables 2a to 2d. The maximum magnitudes for these faults were adopted from the Working Group on Northern California Earthquake Potential (WGNCEP, 1996).

2.2.1 San Andreas Fault

The San Andreas fault extends from the Gulf of California, Mexico, to Point Delgada on the Mendocino Coast in northern California, a total distance of 1,200 km (746 miles). The San Andreas accommodates the majority of the motion between the Pacific and North American plates. This is the largest active fault in California and is responsible for the largest known earthquake in northern California, the 1906 moment magnitude (M_w) 8 San Francisco earthquake (Wallace, 1990). Movement on the San Andreas fault is right-lateral strike-slip, with a total offset of some 560 km or 348 miles (Irwin, 1990). In northern California, the San Andreas fault is clearly delineated, striking northwest, approximately parallel to the vector of plate motion between the Pacific and North American plates. Over most of its length, the San Andreas is a relatively simple, linear fault trace. However, immediately south of San Francisco Bay, the fault splits into a number of fault splays, including the Calaveras and Hayward faults. In the San Francisco Bay area the main trace of the San Andreas fault forms a linear depression along the peninsula occupied by the Crystal Springs reservoirs. Geomorphic evidence for Holocene faulting includes fault scarps in Holocene deposits, right-laterally offset streams, shutter ridges and closed linear depressions (Wallace, 1990). The 1906 earthquake resulted from rupture of the fault from San Juan Bautista north to Point Delgada, a distance of approximately 470 km (293 miles). The average amount of slip on

the fault during this earthquake was 5.1 meters (16.7 feet) in the area to the north of the Golden Gate, and 2.5 meters (8.2 feet) in the Santa Cruz Mountains (WGNCEP, 1996).

Based on differences in geomorphic expression, fault geometry, paleoseismic chronology, slip rate, seismicity, and historic fault ruptures, the San Andreas fault is divided into a number of fault segments. In the San Francisco Bay Area, these segments include the Santa Cruz Mountains, possible source of the 1989 M_w 7.1 Loma Prieta earthquake and the Peninsula segment. The slip rate on these segments is 14 and 17 mm/year (0.55 and 0.67 inch/year) respectively. Both of these segments, along with the North Coast segment, ruptured during the 1906 M_w 8 San Francisco earthquake. The Santa Cruz Mountains, Peninsula, and 1906 rupture segments have calculated maximum credible earthquakes (MCEs) of M_w 7, 7, and 8, respectively (WGNCEP, 1996).

2.2.2 Hayward Fault

The Hayward fault extends for 100 km (62 miles) from the area of Mount Misery, east of San Jose, to Point Pinole on San Pablo Bay. Systematic right-lateral geomorphic offsets and creep offset of cultural features have been well documented along the entire length of the fault (e.g., Lienkaemper et al., 1991). The Hayward fault is considered to be the most likely source of the next major earthquake in the San Francisco Bay area (WGCEP, 1990). The last major earthquake on the Hayward fault, in October 1868, occurred along the southern segment of the fault. This Richter local magnitude (M_L) 7 event caused toppling of buildings in Hayward and other localities within about 5 km (3.1 miles) of the fault. The surface rupture associated with this earthquake is thought to have extended for approximately 30 km (18.6 miles), from Warm Springs to San Leandro, with a maximum reported displacement of 1 meter (3.28 feet). As well as moving during earthquake ruptures, the Hayward fault also moves by a seismic creep. Measurements along the fault over the last two decades show that the creep rate is 5 to 9 mm/year (0.20 to 0.35 inch/year) (Lienkaemper et al., 1991). The Working Group on Northern California Earthquake Potential (1996) assigns MCEs of M_w 6.9 and recurrence intervals of 210 years for both the northern and southern segments of the Hayward fault. Recent research of historical documents has led to the conclusion that an earthquake in 1836, previously thought to have occurred on the northern Hayward fault occurred elsewhere, thereby increasing the time since the last earthquake on this segment of the fault. Recent paleoseismic trenching along the northern Hayward fault indicates that the last surface rupturing earthquake along this part of the fault was sometime between 1626 and 1724 (Lienkaemper et al., 1997). This study also indicated that there are at least four surface rupturing earthquakes in the last 2,250 years.

2.2.3 Hayward Fault - Southeast Extension

The Hayward fault zone extends to the south of Milpitas, along the eastern margin of the Santa Clara Valley, as a broad zone of reverse and reverse-oblique faulting. This zone of faulting includes the Quimby, Crosley, and Evergreen faults, as well as the southeastern extension of the Hayward fault. The predominant sense of motion on these faults is right-lateral strike-slip on the continuation of the Hayward fault itself, becoming reverse-oblique, with an increasing thrust faulting component towards the west. The thrust faults dip to the northeast. Trenching investigations and the geomorphic expression of these

faults indicate that they have been active during late Pleistocene, and possibly even Holocene time. The Working Group on Northern California Earthquake Potential (1996) assigns a maximum earthquake of M_w 6.5 to the southeastern extension of the Hayward fault.

2.2.4 Calaveras Fault

The Calaveras fault is a main component of the San Andreas system, branching off the main San Andreas fault south of Hollister, and extending northwards for approximately 120 km (75 miles) to die out in the area of Danville. The predominant sense of motion on the Calaveras fault is right lateral strike-slip. A smaller component of vertical displacement is evident in some areas along the fault trace. The Calaveras can be divided into two distinct sections, northern and southern, with the boundary located at Calaveras Reservoir.

The Calaveras fault has generated a number of moderate-size earthquakes in historic time, including: (1) the 1861 M_L 5.9 event; (2) the 1886 M_L 5.4 event; (3) the 1897 M_L 6.2 event; (4) a probable M_L 6.5 event in 1911; (5) the 1988 M_L 5.1 Alum Rock event; (6) the 1979 M_L 5.9 Coyote Lake event; and the 1984 M_L 6.2 Morgan Hill event.

To the south of Calaveras reservoir, microseismicity clearly delineates the active trace of the fault. The long term slip rate and contemporary creep rate for the southern Calaveras fault are approximately 15 mm/year (0.59 inch/year) (WGNCEP, 1996). The Working Group for Northern California Earthquake Potential (1996) calculated a MCE of M_w 6½ for the (southern and northern) Calaveras fault.

2.2.5 Monte Vista - Shannon Fault

The Monte Vista-Shannon fault zone is one of a series of southwest-dipping thrust and reverse-oblique faults along the southwestern margin of the Santa Clara Valley. Collectively known as the Foothills Thrust System, this is a series of range-front faults that mark the boundary between the southern Santa Cruz Mountains and the western Santa Clara Valley. The Monte Vista fault extends from Palo Alto, southeast, through Los Altos, Los Altos Hills and Cupertino to Saratoga. Rather than a single fault, this is a belt of southwest-dipping thrust faults. Limited trench data, stream channel profiles, and stream terrace elevations all indicate late Pleistocene and possibly Holocene movement on the Monte Vista thrust (Hitchcock et al., 1994). To the south of Saratoga, the continuation of the Monte Vista fault is mapped as the Shannon fault. The Shannon fault also comprises a series of southwest-dipping thrust faults that offset Miocene bedrock and cut a paleosol reported to be less than 20,000-years-old [R. McLaughlin, U.S. Geological Survey, reported in Hitchcock et al., (1994)]. The Monte Vista-Shannon fault is capable of generating a MCE of M_w 6¾ (WGNCEP, 1996).

2.2.6 Sargent - Berrocal Fault

The Sargent-Berrocal fault is a 94-km-long (58.4 miles), complexly branching system of southwest-dipping thrust and reverse oblique faults associated with the San Andreas fault. Evidence of recent movement along the Sargent fault include reports of surface rupture

along the fault trace in the vicinity of Page Mill Road in Palo Alto during the 1906 earthquake and some possible aftershocks of the 1989 Loma Prieta earthquake. The northern two-thirds of the fault converges with the San Andreas fault at depth, and may not be an independent seismic source (WGNCEP, 1996). The Sargent-Berrocal fault is considered capable of generating a MCE of M_w 6¾.

2.3 GROUND MOTIONS

To characterize the attenuation of ground motions from the seismic sources (described above) to the Guadalupe River Flood Control Improvement Program, we have used empirical attenuation relationships appropriate for deep soil. These include Abrahamson and Silva (1997), Boore et al. (1993), Campbell (1997), and Sadigh et al. (1997). 84th percentile horizontal ground accelerations (PGAs) were calculated for each of the designated four segments of the Guadalupe River Flood Control Improvement Project (Tables 2a to 2d). The limits of the four segments are summarized in the following table.

Segment	From	To
1	Alviso Marina	Tasman Drive
2	Tasman Drive	Montague Expressway
3	Montague Expressway	Highway 101
4	Highway 101	I-880

The PGAs were calculated using the MCE for each seismic source described above, and the closest approach of each of these sources to each segment of the project. For each site, the controlling seismic source is a M_w 6.5 earthquake along the southeast extension of the Hayward fault. This event is capable of generating an 84th percentile peak horizontal ground acceleration of 0.55 g in Segment 1, which includes the levee limits of Task 2.4.4.

2.4 SUMMARY

There are no known active faults that cross the project alignment. However, the levees will be subject to strong ground shaking from known nearby active faults.

SECTION 3

3.1 SURFACE CONDITIONS

3.1.1 Field Reconnaissance

The surface conditions will be described for both the East and West Levees, starting at the north end at the UPRR Bridge (Station 7+008) and proceeding to the south end (Station 8+000) near Route 237. The description of the surface conditions is based on our previous field reconnaissance during which we walked the entire alignment of both levees on December 19, 22 and 23, 1997 from Station 6+100 to Station 17+600. At that time, we observed surface conditions of the crest, both side slopes and the inboard and outboard toe of each levee. Our observations were documented in detail and presented in Appendix D of the April 2000 report. This documentation also included areas of apparent distress such as surface cracking, wet areas, settlement, deformation and undermining of the levee toe. At the time of our field visit, the water flowing in the Guadalupe River was confined to the low flow channel; therefore, the water surface generally was below the level of the inboard toe.

3.1.2 General

In general, the East and West Levees are of earthfill construction, are trapezoidal in cross section and range in height from about 2 ½ to 4 ½ meters (8 to 15 feet). The crest serves as an access road and is covered with armoring aggregate (gravel). In 1995 both levees between the SPRR and Highway 101 were raised, including some areas with soil reinforcement or concrete floodwalls; in many cases the top slopes were steepened. The entire 1995 crest was surfaced with armoring aggregate; the crest width is about 4 to 5 ¾ meters (13 to 14 feet). Numerous culverts and pipes also cross or penetrate the levees. Typically both the inboard and outboard slopes are vegetated with grass, weeds and scattered shrubs. Animal burrows are common along the entire alignment but are randomly located on the face of both side slopes. Access for vehicles is provided by the roadway on the crest and supplemented with ramps cut into the inboard slopes of both levees; at some locations an access road is also positioned just beyond the toe of the inboard slope. The low flow channel in some areas approximates the design centerline and in some areas meanders in close proximity to the inboard toe of the levee. Between the inboard toes of the levees, (i.e., in the channel area), the vegetation typically consists of weeds, grass and occasional trees.

Surface conditions for each segment are described in more detail in the following sections; refer to Appendix D of the April 2000 report for a more complete list of observations, including culverts and pipes observed during a site reconnaissance. For reference purposes for each segment, please refer to the topographic drawings in Figures 3a to 3o in the April 2000 report that include aerial photography.

3.1.3 Segment from UPRR Tracks to Highway 237-Station 7+000 to Station 8+000

Within this segment, three bridges span the river and levees at the UPRR (Station 7+050), Gold Street (Station 7+110) and Route 237 (Station 8+050). The abutments of all three bridges are positioned on both levees.

3.1.3.1 East Levee

Immediately east of Gold Street, there is a widened levee section. Reportedly in 1986 or 1988 during raising of the Gold Street Bridge (refer to Location No. 8 in Table 2 of the April 2000 report), asbestos was encountered in an excavation made in the vicinity of an old meander. Reportedly the asbestos was later capped with about 1.8 meters (6 feet) of silt near the outboard toe; there have been no reported post-construction problems.

A sump pump near Station 7+200 lifts water from an open ditch that collects area surface runoff; the water is pumped through a pipe that penetrates the levee.

A few minor cracks were observed on the crest between about Station 7+150 and Station 7+170; the surface was slightly depressed on the inboard edge.

A wrecking yard is located north of the levee between Gold Street and about Station 7+500. A golf driving range and golf practice fairways are located north of the levee from Station 7+500 to 7+800. A wet spot was observed near the inboard toe at about Station 7+550 and Station 7+700.

Between Station 7+800 and Route 237, the area north of the levee is low and undeveloped. It is covered with a growth of weeds, grass and low shrubs. An apparent old water-filled meander parallels the outboard toe of the levee; a tall growth of brush borders both banks of the meander. Between Station 7+690 to 7+710, minor erosion/undermining of the inboard toe was observed, as well as a wet spot.

3.1.3.2 West Levee

The area south of this levee between UPRR and Gold Street contains some small buildings, houses and an electrical transmission tower. A mobile home park abuts the levee between Gold Street and about Station 7+600.

Prior to 1998, the area between Station 7+600 and Route 237 was undeveloped; most of the area appeared to be level as a result of regrading. Two areas adjacent to the outboard toe were fenced off. One of these areas at about Station 7+630 contained standing water; miscellaneous fill consisting of chunks of asphalt concrete, portland cement concrete and soil was spread around most of the perimeter of this fence. An apparent water filled meander extended from about Station 7+800 to Route 237; the alignment inferred that it might have been part of a larger former meander that continued across both levees. Between approximately Station 7+880 and 7+910 a sag was observed for the full width of the levee crest; the adjacent fresh vegetation on side slopes and new gravel surface suggests it might have been backfill for a new pipeline.

3.1.3.3 Mitigation Levee

In 1999, the Mitigation Levee was constructed west of the existing West Levee and extends north from Route 237 to the West Levee. It ties into the West Levee at about Station 7+580. The crest is covered with gravel, whereas the east slope is covered with grass. An industrial park is located west of the Mitigation Levee. The west slope is covered primarily with wood chips and landscaped with a few bushes and small trees;

some rodent holes were also observed. However, at the north and south ends of the west slope, it is covered with grass.

3.2 SUBSURFACE CONDITIONS

3.2.1 Field Exploration

The subsurface conditions along the East and West Levees were investigated for the 2000 study by drilling 34 exploratory borings (Borings EB-01 through EB-17 and Borings WB-01 through WB-17) and 28 cone penetration tests (CPT) (EC-01 through EC-13 and WC-01 through WC-15) at the locations shown on Figure 2. The results of the previous investigation were presented in Appendix E and Appendix F in the April 2000 report.

For the current study, six additional borings (EB-18 through EB-20 and WB-18 through WB-20) were drilled through the levee embankment, and one additional boring (CB-01) was drilled in the channel. Channel Borings CB-14 and CB-15 are outside of the limits of this study, but are included in Appendix B due to their close proximity of the UPRR bridge. Borings WB-18 through WB-20 were positioned in the Mitigation Levee. Please note that borings drilled and CPTs performed on the East Levee start with the letter E; similarly borings and CPTs on the West Levee start with the letter W, and borings within the channel start with the letter C. A detailed discussion of the techniques used for the subsurface investigation is presented in Appendix B. The depths of these borings ranged from 10.2 to 13.6 meters (33½ to 44½ feet). Figure B-1 presents the Unified Soil Classification System, as well as guidelines summarizing soil consistency and relative density used in preparation of the boring logs. Figure B-2 is a Boring Log Legend that includes the notation used for the types of samples and methods of advancing them. Comprehensive descriptions of the soils encountered at each location are presented on the Logs of Borings, Figures B-3 through B-11. The soil conditions encountered are summarized in Section 3.2.3 "Soil and Groundwater Conditions".

3.2.2 Laboratory Testing

Soil samples were carefully sealed in the field and returned to our laboratory for testing. Soil classifications made in the field were verified in the laboratory after further examination and testing. Laboratory tests were performed on selected soil samples. These tests include, but are not limited to, water content, dry density and unconfined compressive strength, Atterberg Limits, and sieve analysis. The results of these tests are presented at the corresponding sample locations on the Log of Borings. One sieve analysis was conducted to determine the grain size distribution of a selected sand sample. Two Atterberg Limits tests were performed on selected samples of fine-grained soils.

A more comprehensive discussion of the laboratory testing program is presented in Appendix C.

3.2.3 Soil and Groundwater Conditions

We have based our assessment of the soil conditions on three major sources:

- Field exploration conducted for this study (detailed logs of borings are presented in Appendix B);
- Field exploration previously conducted by URS and presented in our April 28, 2000 report;
- Existing subsurface data from previous investigations by our predecessor firms as well as by others (a list of the sources and the boring logs are presented in Table 1); and
- Field exploration conducted previously by another firm for the Mitigation Levee (logs of borings presented in Appendix A).

At the time this report was prepared, surveyed ground surface elevations of borings were not available. Elevations were estimated from nearby cross section geometry provided by the District and from spot elevations provided by CH2MHILL.

We previously prepared two color-coded idealized soil profiles illustrating the subsurface conditions encountered in the 1999 and preceding investigations. These profiles are presented in Figure 5 and Figure 6 for the East Levee and West Levee, respectively, in our April 28, 2000 report. Based on those two figures and logs of seven (7) recent borings, we have prepared the following description of the soil and groundwater conditions.

3.2.3.1 East Levee

Stations 7+008 to 8+000

Levee fill extends to a depth of about 4.5 meters (15 feet). The fill consists of lean to fat clay which contains some sandy lenses and occasional wood fragments. These levee fills are very stiff to hard in consistency, conceivably due to method of placement and desiccation.

Below the levee fill, fat clay extends as deep as 11 meters (36 feet) and consists of Bay Mud with low density and high moisture content. Seams and lenses of fine sand were found throughout this layer.

Other foundation soils consist of lean clay with sand extending to 12.2 meters (40 feet). The clay is medium to stiff in consistency; it is moderately compressible. Underneath this clay layer is a sand deposit extending beyond the depth of most of the previous borings. The sand is medium dense to dense in relative density, and is relatively incompressible.

Groundwater levels measured in the borings for the past 30 years appear to reflect seasonal changes between Elevation -1.2 to 1.5 meters (-4 to 5 feet). In the 1999 investigation, groundwater was encountered in EB-01 at approximately Elevation -2.1 meters (-7 feet).

3.2.3.2 West Levee

Station 7+008 to 8+000

A heterogeneous fill consisting of lean clay to silt forms the levee that extends to approximate Elevation -1.5 to 1.5 meters (-5 to 5 feet). The fill material is very stiff to hard in consistency within the upper 1.5 to 2.1 meters (5 to 7 feet) and becomes medium to soft near the fill and foundation soil interface. Between approximately Station 7+500 and 7+800, a mixture of soft, elastic silt to fat clay occurs below the levee fill and extends to approximately Elevation -9.1 to -10.4 meters (-30 to -34 feet). An old river meander occurred within this reach, so it is conceivable the silt and clay mixture are the "dredged mud" fill from the construction of the 1960 levee improvement project. At Boring WB-03, very soft, lean to fat clay was encountered between approximately Elevation -3.4 to -5.2 meters (-11 to -17 feet). The occurrence of this soft clay layer is consistent with the location of an old river meander; these deposits could be the fine sediments associated with the old river channel.

For the portion of this reach west of Station 7+500, the foundation soil below the levee fill, extending to Elevation -8.2 to -8.8 meters (-27 to -29 feet), consists of lean clay of medium to stiff consistency, and it is considered to be moderately compressible. Below the clay layer, deposits of medium dense silty sand extend beyond approximately Elevation -18.0 meters (-59 feet).

Groundwater levels measured in the 1960's exploration varied between Elevation -0.9 to 0.6 meter (-3 to 2 feet). During the 2000 investigation, no groundwater was encountered above approximate Elevation -1.2 meters (-4 feet).

3.2.3.3 Mitigation Levee

Between approximately Stations 7+570 and 8+000, levee fill in Borings WB-18 through WB-20 extends to a depth of 3.4 to 7.0 meters (11 to 23 feet). The fill consists of generally stiff to hard lean clay; the lower 0.6 meter (2 feet) of fill in WB-19 consists of very dense clayey gravel with pieces of concrete.

Below the levee fill are layers of lean clay, sandy lean clay and fat clay to the depths explored. In general, it is medium to stiff in consistency with some soft pockets in Borings WB-20 at depths of about 9.1 meters (30 feet) and 12.2 meters (40 feet). Roots, stringers and organic odor were noted in the fat clay in Boring WB-18. A few interbeds of loose silty sand and clayey sand ranging in thickness from 0.3 to 2.0 meters (1 to 6½ feet) were encountered within the clay deposits.

Since the rotary wash method was used for the full depth of the borings, groundwater was not measured. However in four previous November 1996 borings by others (Appendix A) along the proposed levee alignment, groundwater was encountered between Elevations -1.2 meters (-4 feet) and -1.8 meters (-6 feet).

3.2.3.4 River Channel

Three Borings CB-01, CB-14, and CB-15 were drilled in the river channel, i.e. between the inboard toes of the East Levee and West Levee. As shown in Figure 2, these borings are positioned near the Gold Street and UPRR bridges. Fill was encountered to depths of 2.3 meters (7½ feet) in Boring CB-01 and 0.9 meters (3 feet) in Borings CB-14 and CB-15; it ranges from soft lean to fat clay to dense clayey sand. In Boring CB-01, the fill is underlain by mostly soft to medium fat clay to the terminal boring depth; a layer of loose sand occurred between depths of 8.2 to 9.5 meters (27 to 31 feet). In Boring CB-14, the fill was underlain by numerous layers of stiff to hard lean clay, dense silty and clayey gravel, soft to medium fat clay and loose silty sand. In Boring CB-15, the fill was underlain by fat clay to clayey sand.

Groundwater was encountered at the ground surface in Boring CB-01 [approximate Elevation 1.5 meters (4.9 feet)] and at Elevation -.5 meter (-1.5 feet) in Boring CB-15.

SECTION 4

4.1 CROSS SECTIONS SELECTED FOR ANALYSIS

During our meeting of November 10, 1998 with the District, we presented two color-coded subsurface profiles (Figures 6 and 7 in the April 2000 report), one for the East Levee and a second for the West Levee. Our interpretation of the soil layers in each profile was based on our evaluation of the subsurface information, including both borings and cone penetration tests (CPTs). Based on these two profiles, we identified a total of 23 levee sections as candidates for analysis along the entire alignment between Station 6+200 and Station 17+500; the levee sections are listed in Table 3. At the request of the District for the 2000 study, we selected and performed engineering analyses for what we believe represented the seven (7) most critical cross sections within the 23 levee sections.

For this current study for Task 2.4.4, we have analyzed six (6) additional cross sections between Station 7+000 and 8+000. A list of the six (6) cross sections is presented in Table 4 and the location of each cross section in plan view is shown in Figure 2. The basis for selection of the six (6) sections is presented below.

4.1.1 Selection of 23 Levee Cross Sections

The two idealized soil profiles (Figure 6 and Figure 7 of the April 28, 2000 report) presented in our November 10, 1998 meeting were based on the logs of 185 previous borings, as well as the 34 new borings and 28 new CPTs for the 2000 study. Based on our engineering judgment and interpretation of these two profiles, we identified 13 levee sections along the West Levee (Sections W1 through W13), and 10 levee sections along the East Levee (Sections E1 through E10). The approximate channel control line station limits of each levee section are listed in Table 3. We also plotted the limits of each levee section in plan view on Figure 2. Although a total of 23 levee sections for both West and East Levees were initially identified, in our opinion three (3) sections of the West Levee (Sections W2, W9 and W10) have similar subsurface conditions as the East Levee (Sections E2, E9 and E10, respectively). Therefore, from an engineering viewpoint, we believe the subsurface conditions along the entire project alignment can be represented by a reduced total of 20 levee sections. Three of the 23 previously identified levee sections (E2, W1, and W2) lie within the limits of this study.

4.1.2 Selection of Six Levee Cross Sections for Task 2.4.4

According to our Agreement, a maximum of six (6) levee cross sections were to be analyzed. We have selected six (6) sections that we believe are reasonably representative of the most critical conditions. Critical is defined as most likely having the lowest factor of safety for slope stability, based on our engineering judgment. The six (6) levee cross sections we selected are shown in Table 4 and are defined as Cross Sections 8 through 13. The reasons for selecting these six (6) are listed under the column entitled "comments" in Table 4; they vary from the presence of soft, fat clay (CH), to very loose sand in the levee foundation, to steep side slopes and a ditch at the outboard toe, to a new levee. The locations of each cross section are shown in plan view in Figure 2.

Initially, one critical cross section was chosen from each original levee section identified with similar soil conditions. For comparison, the average side slope inclinations and slope heights were tabulated. Selection of the critical cross section in each levee section was based on slope inclination and slope height. Slopes with steeper side slopes or larger heights were judged to have a lower factor of safety. Additional cross sections were then selected where deemed necessary by project scope or engineering judgment.

The levee improvements (as indicated by the plans prepared by CH2MHILL, dated August 2001) were incorporated into the selected critical cross sections. The proposed geometry, including levee raise and excavation beyond the inboard toe, was modeled in the slope stability and seepage analyses. The design high water level was obtained for each cross section from the plans prepared by CH2MHILL.

Soil conditions were also a factor in the selection of the critical cross sections. Soft clay and loose sand layers are considered critical. The presence of soft, fat clay (CH) in the levee foundation was identified using the idealized soil profiles previously presented as Figures 6 and 7 of the 2000 report. Soft foundation soils were found in the critical cross sections selected based on geometry. Locations of loose sand indicated on Figure 46 of the 2000 report, and indicated by blow count data within the levee section, were also reviewed for selection.

We prepared two drawings for each of the six (6) cross sections: (1) one drawing showing a cross section representing the soil conditions and existing levee geometry, and a table summarizing the selected geotechnical design parameters (Figures 5 through 10); and (2) a second drawing showing four graphs where the undrained shear strength (S_u), moisture content, dry density and blows per foot are plotted with respect to elevation (Figures 11 through 16). The latter drawing includes data from all available borings in both levees within the station limits of the relevant levee section. The District furnished all original geometric levee sections; levee improvements were then incorporated based on the plans prepared by CH2MHILL.

4.1.3 Selection of Shear Strength Parameters

At the six levee cross sections, shear strength parameters were selected for each soil layer based on the field and laboratory data. The selected design parameters are summarized in the upper left-hand corner table of Figures 5 through 10 for Cross Sections 8 through 13, respectively. These methods are described in more detail as follows.

4.1.3.1 Cohesive Soils in Levee and Levee Foundation

Undrained Strength

As an example, refer to Cross Section 8 on Figures 5 and 11 where three cohesive soil layers are shown in the levee fill and foundation including:

- Layer 1 is Lean Clay fill (CL), about 4.0 meters (13 feet) thick
- Layer 2 is Fat Clay (CH), about 3.4 meters (11 feet) thick

- Layer 3 is Lean CLay (CL), about 5.5 meters (18 feet) thick

As shown in S_u versus elevation in Figure 11, all of the unconfined compression, undrained unconsolidated triaxial shear strength tests (UU) and CPT test results within the limits of levee Section E2 are plotted. Based on this plot for Layer 1 (Elevation 4.6± 0.6 meter or Elevation 15 to 2± feet) an S_u value of 73.7 kPa (kilo Pascals) [1,540 pounds per square foot (psf)] was selected as represented by the solid vertical line. For Layer 2 (Elevation 0.6 to –2.7 meters or Elevation 2 to -9± feet) an S_u value of 86.2 kPa (1,800 psf) was selected. For Layer 3 (Elevation –2.7 to -8.2 meters or Elevation -9 to -27± feet) an S_u value of 57.5 kPa (1200 psf) was selected. These values were then entered in the table at the upper left-hand corner of Figure 5. It should be noted that the S_u value selected is not a lower bound; in some cases the S_u values plotted are believed to underestimate the soil strength because of the presence of sand interbeds in the samples tested.

In a similar manner, the moisture content and dry density for each layer in Cross Section 8 were obtained from the other plots in Figure 11 and entered into the table in the upper left hand corner of Figure 5.

The embankment strengths were reduced to account for progressive failure in embankments on soft clay foundations. Since the embankment (fill) shear strengths were considered to be relatively high, the embankment strengths were reduced according to the procedures outlined by Duncan and Buchignani (1975). The plot used to determine the embankment strength reduction factor *(R_E)* is presented in Figure 17. For the foundation, we used a foundation strength reduction factor of 1.0 for Type A soil, also presented in Figure 17.

Effective Strength

The data from consolidated undrained (CU) triaxial shear strength tests with pore pressure measurements are presented in Table 5. These test results include both existing tests and new tests performed for this current investigation; they are divided into the following five categories:

- Lean clay fill (CL), dry density < 1,762 kilograms per cubic meter (kcm) [110 pounds per cubic foot (pcf)
- Lean clay fill (CL), dry density > 1,762 kcm (110 pcf)
- Fat clay (CH), dry density < 1,281 kcm (80 pcf)
- Lean clay (CL), 1,281 kcm (80 pcf) < dry density < 1,602 kcm (100 pcf)
- Lean clay (CL), 1,602 kcm (100 pcf) < dry density < 1,762 kcm (110 pcf)

Two effective strength parameters are shown in Table 5 including ϕ' (angle of internal friction) and c' (cohesion); values are listed for peak stress, 10 percent and 20 percent strain criteria. A design value was selected between the peak stress and 10 percent strain; in our judgment 20 percent strain is too large a deformation for the levee to undergo. The value from Table 5 was then entered into the upper left-hand corner table of Figure 5 for

Layers 1 through 3. For example, Layer 1 was lean clay fill with a dry density of 1,746 kcm (109 pcf); from Table 5 for a dry density less than 1,762 kcm (110 pcf), $\phi' = 24$ degrees and $c' = 21.5$ kPa (450 psf) were selected.

Table 5 was prepared based on the graphical CU test results plotted in Figures 18 through 22. For example for the lean clay fill category with samples with dry density less than 1,762 kcm (110 pcf) for peak stress, all q versus p values were plotted in Figure 18. The q and p values are defined as follows:

- $q = \frac{1}{2}(\sigma_1' - \sigma_3')$
- $p = \frac{1}{2}(\sigma_1' + \sigma_3')$

The best fit straight line (method of least squares) was then plotted in Figure 18 for peak stress and the ϕ' and c' values calculated (as shown in that figure) as 23.7 degrees and 21.8 kPa (456 psf), respectively.

4.1.3.2 Cohesive Soil in Channel

Undrained Strength

Borings were advanced in the channel (beyond the inboard toe of the levee) as part of this study in order to assess the strength of the materials subjected to periods of wetting and riverbed deposition. In general, the undrained strengths of the channel sediments were lower than undrained strengths of soils within the levee or levee foundation.

The extent of the channel sediments was estimated by extending a ½:1 (horizontal to vertical) line beneath the inboard levee toe. Cohesive soil inboard of this line was assumed to have channel sediment properties. Cohesive soil outboard of this line was assumed to have the properties determined from the index property plots of levee and foundation soils.

For Cross Sections 8 through 13, the dry density, moisture content, and undrained strength of the channel sediments were estimated from the results of dry density, moisture content, and unconfined compression tests performed on samples from Borings CB-01, CB-14, and CB-15. Although Borings CB-14 and CB-15 are located slightly outside the geographical limits of Station 7+008 to 8+000, it is our engineering opinion that the river sediments are similar to those at Boring CB-01. The plots of undrained strength, moisture content and dry density versus depth for the channel sediments are presented in Figures 11 through 16. Selected design parameters for the channel sediments are represented by a dashed vertical line and the values were inserted in the tables in Figures 5 through 10. The selected undrained strength was estimated using the results of laboratory testing, local experience, and engineering judgment.

Effective Strength

The effective strengths of the cohesive channel sediments were determined in the same manner as outlined for cohesive soils in the levee and levee foundation.

4.1.3.3 Granular Soils

Effective strength parameters were estimated by the following method (refer to Layer 4, silty sand, in the upper left-hand corner table of Figure 5). The effective stress σ_v' at the middle of Layer 4 was calculated to be about 143.6 kPa (3,000 psf). For a Standard Penetration Test N value of 10 and σ_v' of 143.6 kPa (3,000 psf), the estimated relative density (D_r) is about 50 percent (see Figure 23). According to a published graph (see Figure 24) by NAVFAC (1982) for a silty sand and D_r of 50 percent, and an approximate dry unit weight of 105 pounds pcf, ϕ' is about 33 degrees.

In conclusion, it is our opinion the six sections selected and listed in Table 4 (Cross Sections 8 through 13) are reasonably representative of the subsurface conditions that may be critical from a levee stability and design viewpoint. We performed slope stability analyses at each of these sections based on the parameters summarized in Figures 5 through 10 for the existing levee geometry with the improvements as specified by CH2MHILL.

4.2 METHOD OF ANALYSIS

Four design conditions were analyzed for the slope stability evaluation in accordance with the Corps Manual for the Design and Construction of Levees (EM 1110-2-1913). The conditions, the slopes analyzed, shear strength guidelines, and minimum factors of safety required for design are listed below.

Case	Design Condition	Slope Analyzed	Required Minimum Factor of Safety
I	End of Construction	Riverside and Landside	1.3
II	Sudden Drawdown	Riverside	1.0
III	Steady Seepage from Full Flood Stage	Riverside and Landside	1.4
IV	Earthquake	Riverside and Landside	Not applicable

As outlined in the Corps guidelines, earthquake loadings are not considered in analyzing the stability of levees because of the low probability of an earthquake coinciding with a period of high water. Nevertheless, we evaluated the seismic stability of the levee in the following manner because of the close proximity to major fault systems such as the San Andreas and Hayward:

- Performed pseudo-static slope stability analysis using a seismic coefficient approximately one-half of the peak ground acceleration (0.3 to 0.33); this is consistent with the California Department of Conservation, Division of Mines and Geology, Special Publication 117, "Guidelines for Evaluating and Mitigating Seismic Hazards in California," (1997).
- Performed simplified deformation analysis for cross-sections exhibiting a pseudo-

static slope stability safety factor less than 1.0 (Makdisi and Seed, 1977) to estimate the magnitude of potential permanent deformation due to strong earthquake shaking.

The stability of the levees under Cases I, III and IV (pseudo-static) was evaluated using a limit equilibrium method based on Spencer's procedure of slices as coded in the program SLOPE/W (Geo-Slope, Int., 1992). In Spencer's procedure, all forces are assumed to have the same inclination, and all requirements for static equilibrium are satisfied. The trial and error solution involves successive assumptions for the factor of safety and side force inclination until both force and moment equilibrium are satisfied. In addition, the slope stability results of some cross sections computed by SLOPE/W also were evaluated using a simplified chart method. The failure surfaces identified by SLOPE/W using Spencer's procedure are circular. For Case II, we used UTEXAS3, which is also a limit equilibrium computer program based on Spencer's procedure, to evaluate the factor of safety against sudden drawdown loading. This computer program is coded with the 3-stage computation that is appropriate for the sudden drawdown loading condition.

4.2.1 Pore Pressures

The SEEP/W program (Geo-Slope International, 1992) was used to estimate the phreatic surface in the levee for the critical flood stage under the steady seepage condition based on the design water levels provided in the plans prepared by CH2MHILL. The SEEP/W program and the seepage analyses are described in further detail in Section 6 "Seepage Evaluation" of this report. For a given cross section and design condition, the wetted face (phreatic surface) was estimated. The phreatic surface generated by SEEP/W was imported into SLOPE/W to calculate the pore pressures within the levee for Cases II and III.

The phreatic surface at the steady seepage case represents the worst-case groundwater condition. Based on the design hydrograph in Figure 3, we developed a time rate of change of the river level (at a given cross section) in order to assess the advancement of the wetted face into the levee during high flow stages. The hydrograph was discussed previously, in Section 1.1 "Project Description". A transient seepage analysis was performed, which demonstrated that the wetted face into the levee never reached the height of the steady seepage condition; therefore the assumption of the piezometric surface is judged to be conservative.

4.2.2 Static Loading Conditions

The static loading conditions for slope stability analysis prescribed by the United States Army Corps of Engineers (Corps) are listed below.

- End of Construction
- Sudden Drawdown
- Steady Seepage from Full Flood Stage

A discussion of the conditions for analysis and their respective strength parameters is found in the following sections.

4.2.2.1 End of Construction

The end of construction case represents the condition immediately following construction when excess pore pressures have been induced by construction, and not enough time has passed for the pressures to dissipate. For this reason, undrained conditions exist for an impervious embankment and foundation. Strength results from laboratory unconsolidated-undrained tests were used for the fine grained soils. The effective friction angle was used for granular soils, as they are assumed to be pervious and free draining.

4.2.2.2 Sudden Drawdown

The sudden drawdown case represents the condition whereby a prolonged flood stage saturates at least the major part of the upstream (riverside) embankment portion and then drops faster than the soil can drain. This causes the development of excess pore pressures that may result in the riverside slope becoming unstable.

For this analysis, a composite envelope was used for the design strengths for impervious soils. The strength envelope consisted of the drained strengths until the intersection of the undrained strength was reached. At that point and beyond, the undrained strength was used. These strengths are in accordance with the Corps Manual.

4.2.2.3 Steady Seepage From Full Flood Stage

The steady seepage case represents the condition where the water remains at or near full flood stage so that the embankment becomes fully saturated and a condition of steady seepage occurs. The design shear strength should be based on the effective stresses; this should correspond to the residual strengths where previous shear deformation or sliding has occurred.

For this analysis, the effective stress envelope is based on consolidated undrained test results. Recognizing the non-linear nature of the shear strength envelope for higher effective stresses, we have modified the strength using a composite envelope. Drained strengths were used until they became greater than the undrained strengths. At this point and beyond, an envelope midway between the drained and undrained strengths was used. The water levels used are at the design high levels as specified on the plans prepared by CH2MHILL.

4.2.3 Earthquake Loading Conditions

The earthquake case represents the condition when the levee is subjected to earthquake motion. As discussed above, we performed a 2-step evaluation for the earthquake loading. In the first step, we selected a seismic coefficient equal to one-half the peak horizontal ground acceleration as outlined in the 1997 Guidelines for Evaluating and Mitigating Seismic Hazards in California. The peak ground acceleration was determined using a deterministic approach where no consideration is given to the probability of an earthquake occurring. The maximum earthquake magnitudes adopted by the Working Group on Northern California Earthquake Potential (1996) were used for each of the seismic sources. Empirical attenuation relationships were used to calculate the 84th percentile horizontal ground accelerations as discussed previously in Section 2.3 "Ground

Motions". Accelerations were determined for four segments along the reach of the Lower Guadalupe River, as presented in Table 2a through 2d. For the portion of the alignment analyzed in this study (Task 2.4.4), the design seismic coefficient is 0.55 g for Cross Sections 1 and 8 through 13. The strength parameters used for the earthquake condition were the same as for the end of construction. In the second step, we estimated the permanent deformation at cross-sections with a safety factor less than 1.1 under pseudo-static loading analyzed in the first step.

4.3 STABILITY RESULTS OF LEVEE SECTIONS

The minimum (critical) factors of safety computed using the various methods of analysis are summarized in the table below.

		Factor of Safety						
Case	Design Condition	Minimum Required	Cross Section 8 Minimum	Cross Section 9 Minimum	Cross Section 10 Minimum	Cross Section 11 Minimum	Cross Section 12 Minimum	Cross Section 13 Minimum
I	End of Construction (Riverside)	1.3	3.0	3.2	3.5	2.8	9.2	2.5
I	End of Construction (Landside)	1.3	5.0	NS	NS	3.6	12.6	7.3
II	Sudden Drawdown (Riverside)	1.0	1.8	1.8	2.4	1.7	1.4	1.5
III	Steady Seepage from Full Flood Stage (Riverside)	1.4	4.0	3.0	4.8	3.4	3.4	3.0
III	Steady Seepage from Full Flood Stage (Landside)	1.4	2.0	NS	NS	1.9	3.5	4.2
IV	Pseudo-Static Earthquake (Riverside)	1.0	1.1*	1.3	1.2*	1.0	3.6*	1.0
IV	Pseudo-Static Earthquake (Landside)	1.0	1.8	NS	NS	1.8	5.3	2.9*

NS = No Slope

* End of Construction critical circle analyzed with Bishop and Ordinary Methods

The approximate locations of the critical slip surfaces are shown on Figures 5 through 10. The analyzed sections appear to be stable for each of the design conditions.

For a pseudo-static safety factor less than 1.1, we have estimated the permanent

deformation that could occur under the maximum credible earthquakes. The following table presents a summary of our estimates:

Cross-Section Number	Yield Acceleration (g)	Estimated Average Total Deformation (mm)	Estimated Average Vertical Deformation (mm)
8	0.25	160	110
9	0.26	130	90
10	0.28	110	80
11	0.26	130	90
12	0.76	0	0
13	0.27	120	80

The table above presents results in terms of total deformation (Δ_{total}) along the potential sliding surface and estimated vertical displacement ($\Delta_{vertical}$). For the potential sliding surface through or near the crest of the embankment, $\Delta_{vertical} = 0.7\ \Delta_{total}$ was estimated based on the slope of the potential failure surface at the crest. As shown, we estimate vertical deformation in the range of 80 to 110 mm (3 to 4 inches) for Sections 8 through 13, except at Section 12 where movements are estimated to be nil. At all locations we anticipate that shaking from a major earthquake will result in some cracking of the levee, requiring some minor regrading of the levee.

4.4 STABILITY RESULTS OF DEPRESSED MAINTENANCE ROADS

According to Sheet C-4 of the 90-Percent Design Submittal, a depressed maintenance road is planned south of Gold Street at the riverside toe of both levees. Planned side slopes in this area are 2 to 1 (horizontal to vertical). In particular, sediment removal will result in a deepened channel at the riverside toe of the lower maintenance road. The minimum (critical) factors of safety were computed using the methods of analysis previously described. The approximate locations of the critical slip surface are shown on Figures 25 through 31. At these locations we estimated the factor of safety for end of construction assuming heavily loaded trucks for full width of the depressed maintenance road and full width of the levee crown. The surcharge load of the trucks was assumed to be equal to a uniform pressure of 0.61 meter (2 feet) of soil or 12.5 kPa (260 psf). The results are summarized in the table below.

Cross Section		Figure	Factor of Safety End of Construction		
Station	Levee		Minimum Required	Riverside	Landside
7+200	West	25, 26, 27	1.3	3.0/2.3	7.1
7+200	East	28, 29, 30	1.3	2.2/2.1*	2.8
7+260	East	31	1.3	2.1	-

* Factor of safety using Bishop method

More specifically the cross section in Figure 25 shows the critical deep slip circle through the upper maintenance road, whereas the cross section in Figure 26 represents the critical shallow slip circle through the lower maintenance road; both slip circles are on the

riverside West Levee at Station 7+200. The location of the landside critical circle at this same levee and station is shown in Figure 27. For the East Levee at Station 7+200 the cross section in Figure 28 presents the critical deep slip circle through the upper maintenance road, whereas the cross section in Figure 29 represents the critical shallow slip circle through the lower maintenance road. At this same levee and station the location of the landside critical circle is shown in Figure 30. For the East Levee at Station 7+260, the cross section in Figure 31 presents the critical shallow slip circle through the lower maintenance road.

The analyzed sections appear to be stable for the end of construction (EOC) condition. Considering that (1) factors of safety for EOC are well above the minimum required and (2) surcharge loading for other loading conditions is not warranted, it is our opinion that further analysis of these cross sections is not required.

SECTION 5

5.1 SUBSIDENCE

From 1916 to 1966 in the San Jose area of Santa Clara Valley, the artesian head declined from 55 to 73 meters (180 to 240 feet) in response to groundwater withdrawal. As a direct result of the artesian head decline, the land surface subsided as much as 3.9 meters (12.7 feet) in San Jose (Poland and Ireland, 1985). The recovery of artesian head since 1967 has been substantial. In downtown San Jose, the artesian head recovered 21.3 to 30.5 meters (70 to 100 feet) in the 8 years to 1975. The subsidence appears to have been stopped by raising the artesian head in the aquifers. From 1934 to 1960, maximum subsidence of 1.5 meters (5 feet) occurred in San Jose and Sunnyvale. From 1960 to 1967, maximum subsidence of 1.1 meters (3-1/2 feet) occurred in San Jose and Santa Clara; the average rate of subsidence was 152 mm (1/2 foot) per year, the most rapid rate of subsidence in any 7-year period. Based on the land subsidence contour map prepared by United States Geological Survey (Poland and Ireland, 1967), the segment of Guadalupe River within the scope of study experienced subsidence ranging from 1.8 to 2.4 meters (6 to 8 feet) between the period of 1934 and 1967. Since 1967, subsidence has been significantly reduced. In San Jose Wells 7S/1E-16C6 and C11, an extensometer recorded compression decreasing from 305 mm (1 foot) in 1961 and 1962 to 76 mm (1/4 foot) in 1967 and to 3 mm (1/100) foot in 1973. Based on the referenced contour map, a maximum drop of 1.1 meters (3½ feet) in elevation of the levees since their improvement construction in 1960 probably can be attributed to land subsidence.

5.2 LEVEE SETTLEMENT

5.2.1 Inferred Settlement

Based on review of the (1) as-built drawings for the 1960 and 1963 improvement construction of the levees and (2) those for the 1985 and 1995 levee raising and widening of the Guadalupe River channel, we have estimated the levee settlement between Station 7+008 and Station 14+500. Estimates of the settlement are inferred because they are primarily based on the change in elevation of the crest of the levees. No settlement monitoring was performed. Our review indicates that the lowering of crest elevations occurred mostly in areas north of Station 12+000 (Montague Expressway). The levee portions south of Station 12+000 appear to have experienced little or no change in crest elevation. We estimate that the portions of levees north of Station 12+000 settled due to consolidation approximately 229 to 762 mm (average of 381 mm) or ¾ to 2½ feet (average of 1¼ feet) for the West Levee and 396 to 1189 mm (average of 933 mm) or 1¼ to 4 feet (average of 3 feet) for the East Levee for the period of 1960 to 1983. These inferred consolidation settlements deduct the groundwater withdrawal related land subsidence settlement estimated from 1960 to 1967. North of Station 12 +000, we estimated the subsidence settlement to be 457 to 762 mm (1½ to 2½ feet) during this 7-year period.

5.2.2 Predicted Settlement

These estimates have been compared to settlement calculations based on consolidation test results from 1960 and 1999 studies and our current investigation (2002). It is

assumed that the height of the fill at each stage of construction was added instantaneously. More specifically, the construction stages are described as follows.

Cross Section	Station	Levee	Fill Thickness (feet)			
			1963	1983	1995	1999
8*	7+170	East	16.0	5.0	0	-
9*	7+170	West	10.5	2.0	0	-
10	7+320	West	5.0	2.5	0	-
11	7+800	East	11.0	6.0	1.0	-
*12**	7+950	Mitigation	0	0	0	20
13*	7+997	East	20.0	5.5	1.5	-

* Through channel of old meander.
** Mitigation levee constructed in 1999.

The locations of the cross sections are shown in plan view in Figure 2, Site and Field Exploration Location Plan. These cross sections are the same cross sections used in the levee slope stability analysis described in Section 4 "Levee Slope Stability Evaluation". The time rate of settlement for the six (6) cross sections are presented graphically on Figures 32 to 37 for the period from 1963 to 2010. The following table summarizes the estimated consolidation settlements from 1963 to 2003.

Cross Section	Estimated Consolidation Settlement					
	From 1963 to 1983		From 1963 to 1995		From 1963 to 2003	
	(mm)	(inches)	(mm)	(inches)	(mm)	(inches)
8	684	27	802	31½	802	31½
9	387	15¼	440	17¼	440	17 ¼
10	269	10½	435	17¼	435	17¼
11	449	17¾	605	23¾	629	24¾
12	0	0	0	0	732*	28¾*
13	764	30	870	34¼	892	35¼

* From 1999 to 2003.

Cross Sections 8 through 13 are located north of Route 237 (Station 8+000). The values shown in the above table are consistent with those estimated (inferred) from the as-built drawings. For example, the range of estimated consolidation settlement from the above table for the West Levee for the period 1963 to 1983 ranged from 269 (Cross Section 10) to 387 mm (Cross Section 9). As described in Section 5.2.1 "Inferred Settlement" the

inferred consolidation settlement for the West Levee was 381 mm. Furthermore, the range of estimated consolidation settlement from the above table for the East Levee for the period 1963 to 1983 ranged from 449 mm (Cross Section 11) to 764 mm (Cross Section 13). As described in Section 5.2 1 "Inferred Settlement" the range of inferred consolidation settlement for the East Levee was 396 to 1,189 mm. Therefore, we have concluded the approach and methodology used in our settlement analysis are reasonable for both levees.

5.3 FUTURE SETTLEMENT

The levees at each of the six (6) cross sections will undergo settlement in the future from ongoing consolidation of the foundation clay soil due to fills placed in 1963, 1983, 1995 and 1999. Consequently, we have estimated the following long term consolidation settlement for the six sections:

Cross Section	Estimated Future Settlement	
	(mm)	(inches)
8	< 13	< ½
9	< 13	< ½
10	< 13	< ½
11	< 13	< ½
12*	< 13	< ½
13	< 13	< ½

* Mitigation Levee.

Therefore, the future settlement is estimated to be on the order of less than 13 mm (1/2 inch) for West, East and Mitigation Levees.

We understand the Mitigation Levee (Cross Section 12) was constructed with sufficient freeboard to accommodate the 100-year flood. Therefore, a floodwall is not planned at the Mitigation Levee, except at the very west end; details are not available. At the Mitigation Levee (Cross Section 12), the estimated consolidation settlement from 1999 forward is shown in the following table.

Estimated Consolidation Settlement							
From 1999 to 2001		From 1999 to 2003		From 1999 to 2004		From 1999 to 2010	
(mm)	(inches)	(mm)	(inches)	(mm)	(inches)	(mm)	(inches)
732	28¾	738	29	738	29	738	29

According to sheet C-6A, the existing ground surface at the VTA Mitigation Levee ranges from Elevation 6.3 to 6.5 meters. Based on drawings C-3 through C-6 in Appendix A, the crest of most of this levee was built to Elevation 6.3 meters (18.0 feet). This would infer that no settlement has occurred to date; however, if the levee was overbuilt during construction or raised in the meantime, the inferred settlement would

have occurred. In summary, we believe the future settlement at the Mitigation Levee is less than 13mm (1/2 inch).

Between the UPRR and Route 237, some fill will be added behind the two (2) new MSE walls and behind six (6) new floodwalls. At these wall locations, maximum fill height planned is about 1.3 meters (4.3 feet). Additional future settlement is anticipated where new fill is to be placed behind these walls; the estimated magnitude and rate of settlement of those new fills at the new walls are presented later in Section 7.0 "Floodwall, Wingwall and MSE Wall Design Parameters".

SECTION 6

6.1 METHOD OF ANALYSIS

The flood levels used in the analysis of each of the selected sections were provided by CH2MHILL on the plans dated August 2001 and April 2002. The flood levels are summarized as follows:

Cross Section	Station (meters)	100-year Flood Elevation		
		(meters, NAVD*)	(meters, NGVD**)	(feet, NGVD)
8	E 7 + 170	5.06	4.25	13.9
9	W 7 + 170	5.06	4.25	13.9
10	W 7 + 320	5.09	4.28	14.0
11	E 7 + 800	5.30	4.49	14.7
12	W 7 + 950	5.39	4.58	15.0
13	E 7 + 997	5.64	4.83	15.8

* From August 2001 Drawings, CH2MHILL
** Converted to NGVD Datum

Seepage through the levee during maximum flood was evaluated using a steady state analysis. A finite element program, SEEP/W [developed by Geo-Slope International (1992)], was used to perform seepage calculations. SEEP/W is a computer program of two-dimensional code with isoparametric and higher order finite elements capable of solving saturated and unsaturated flow conditions using steady state and transient analysis. The program can represent multiple soil types with anisotropic hydraulic conductivity; it can also represent prescribed constant and transient boundary conditions.

The two boundary conditions established for the steady state analysis are constant head and no flow conditions. The nodes beneath the water on the inboard side of the levee used the boundary condition of head equal to the height of the design water level. A boundary condition of no flow was established across the bottom of the finite element mesh. The nodes on the upstream portion of the levee were assigned a flux-type review flow boundary condition. In this case, after the heads are computed for all nodes, the review nodes are modified if any one of the review nodes has a computed head greater than the y-coordinate of the node (that is, if the porewater pressure is greater than zero). Using the maximum pressure review option, the review node with the maximum positive pressure head is identified by SEEP/W. The head at this node is then set to the y-coordinate for the next iteration. At the outboard boundary of the mesh, a constant head boundary condition was established. The constant head value was inferred from water level observed in historical borings in the vicinity of the cross section.

6.2 CROSS SECTIONS AND RESULTS OF ANALYSIS

In general the levees are constructed of well compacted lean clays or lean to fat clays. Furthermore, most of the native soils directly underlying the levee fills are lean to fat clays. Since the clayey soils are relatively impervious, it is our engineering opinion that

generally (1) seepage through and underneath the levees during the design flood will be relatively small and (2) the potential for piping at the outboard toe will also be very low. To confirm these opinions, we performed a seepage analysis on the six critical cross sections. These cross sections correspond to the cross sections listed and described in Section 4.1 "Cross Sections Selected for Analysis."

The coefficients of hydraulic conductivity (permeability) for three types of soil (lean clay, silty sand and sand) were selected from the following laboratory and published data. They included laboratory permeability test results of relatively undisturbed and remolded samples, broad empirical relationships between soil types and permeability (Cedergren, 1967) as shown in Figure 38, and correlation relationships based on grain size distribution as described in the EM-1110-2-1913 (COE, 1978) and the NAVAC DM-7.1 (1982). Permeability tests were performed for the 2000 study on three relatively undisturbed soil samples as reported in Appendix G of that report. The coefficients of hydraulic conductivity (permeability), k, determined from those samples, are as follows:

Sample Number	Depth (feet)	Soil Type	Location	Dry Density (pcf)	k (cm/second)
EB-03-16-4	30 – 31.5	Lean Clay	Foundation	101.5	3.3×10^{-7}
EB-11-9-4	14.5 – 16	Silty Sand	Foundation	90.3	5.1×10^{-4}
WB-10-11-4	26 – 27.5	Silty Sand	Foundation	125.2	1.2×10^{-4}

The values assigned to the corresponding soil types are presented in the following table. The ratio of anisotropy of the horizontal to vertical conductivity was assumed to be 4 for all soil types.

Soil Type	Coefficient of Hydraulic Conductivity (k_h)		Selected Values	
	cm/sec	ft / hr	cm/sec	ft / hr
Sand	10^{-2} to 10^{-4}	1.2 to 0.012	10^{-2}	1.2
Silty Sand, Clayey Sand	5×10^{-3} to 5×10^{-4}	0.6 to 0.06	6×10^{-3}	0.7
Lean to Fat Clay	10^{-5} to 10^{-6}	1.2×10^{-3} to 1.2×10^{-4}	10^{-5}	1.2×10^{-3}

The six cross sections described above are shown on Figure 39 through Figure 44 for the steady state seepage condition. These figures also present the estimated flow quantities in cubic feet per hour per linear foot for a steady state seepage condition with river levels at 100-year elevations, as summarized below:

		Estimated Seepage Quantity	
Figure Number	Cross Section	Cubic Feet per Hour per Linear Foot	Gallons per Minute per Linear Foot
39	8	2.87E-03	3.58E-04
40	9	4.95E-03	6.17E-04
41	10	2.77E-03	3.45E-04
42	11	1.81E-03	2.26E-04

		Estimated Seepage Quantity	
Figure Number	Cross Section	Cubic Feet per Hour per Linear Foot	Gallons per Minute per Linear Foot
43	12	2.31E-03	2.88E-04
44	13	3.20E-03	3.99E-04

For a 30.5-meter (100-foot) long section, the total flow quantity through the levee at each of the above six sections is expected to be less than 1/10 gallon per minute (gpm). In our opinion these seepage quantities are very small.

The above discussion is for a steady state condition such that seepage through the levee achieves equilibrium during the 100-year flood elevations. To simulate the change in river level during the flood event, we performed a transient analysis based on the 100-year flood hydrograph (Figure 3). The results of the transient analysis are presented in Figure 45 through Figure 50. These figures present the phreatic surface through the levee at different times. For example, in Figure 48, subsequent phreatic surfaces at times of 12 hours (time step 6), 24 hours (time step 12), 36 hours (time step 18), 48 hours (time step 24), 60 hours (time step 30) 72 hours (time step 36), and 84 hours (time step 42). As shown on Figure 48, the highest phreatic surface is at 36 hours (time step 18); and this surface is lower than the steady state phreatic surface shown in Figure 42. This observation is similar to the other cross sections. By comparison, we conclude that the estimated seepage quantities will be much less than the 1/10 gpm estimated previously. However, it should be emphasized that the quantity of estimated seepage refers to the underground water flow through a theoretical cross-section at the toe, i.e. no seepage exiting the toe of the levee.

In Figures 45 through 50, the cross sections indicate thick layers of lean clay soils at the outboard toe of the levee as summarized in the following table. Also shown in the table below is the estimated depth of high groundwater (from transient analysis) below ground surface at the outboard toe of the levee.

Cross Section	Thickness of Lean Clay Layer *		Depth of Groundwater Below Ground Surface*	
	feet	meters	feet	Meters
8	29	8.8	6	1.8
9	31	9.5	8	2.4
10	28	8.5	8	2.4
11	15	4.6	2	0.6
12	26	7.9	9	2.7
13	25	7.6	5	1.5

* At outboard toe of levee

Based on the above table, the thickness of lean clay layer generally ranges from 7.6 meters (25 feet) to 9.5 meters (31 feet); an exception is Cross Section 11 where the thickness is 4.6 meters (15 feet). Also based on the above table, the phreatic surface never exits at the toe ground surface and generally ranges from 1.5 meters (5 feet) to 2.7 meters (9 feet); an exception is Cross Section 11 where the depth to phreatic surface is

about 0.6 meter (2 feet). Due to the combination at the outboard toe of (1) thick layers of cohesive soils, (2) cohesiveness of these soils, and (3) the moderate depth to phreatic surface, piping is unlikely.

SECTION 7

7.1 BACKGROUND

As noted in the July 2002 Design Drawings prepared by CH2MHILL, the levee improvements between Station 7+008 and Station 8+000 include the construction of seven (7) reinforced concrete floodwall on both the East Levee (FW2E, FW3E and FW4E) and West Levee (FW1W, FW2W, FW3W and FW5W). The lack of right-of-way needed to raise the levees in this particular area has dictated the need for floodwalls. The floodwalls vary in height (above existing ground surface or crown) from 0.45 to 1.95 meters (1.5 to 6.4 feet). Additional fill height ranging from 0.9 to 1.5 meters (3 to 4.9 feet) is planned behind floodwalls FW2E, FW3E, FW1W, FW2W, FW3W and FW5W. The vertical floodwall will be positioned with the vertical face offset 0.31 meter (1 foot) from the inboard hinge point of the crest; this will result in access to a full width maintenance road even during the 100-year design flood. The 0.31 meter (1 foot) offset allows for irregularities (such as levee erosion), thereby optimizing the life span of the floodwall.

We provided preliminary design parameters for the floodwalls in our April 14, 2000 and October 23, 2001 letters to the District and to CH2MHILL, respectively. A copy of each letter is presented in Appendix D. These parameters are defined by reach as designated by CH2MHILL. As discussed in Section 1.1 "Project Description", only Reach G (Station Limits from 7+008 to 8+000) applies to Task 2.4.4. Therefore the following discussion applies to Reach G.

As shown in the July 2002 Design Drawings, the levee improvements between Station 7+008 and Station 8+000 also include the construction of two MSE walls, MSW1E (on East Levee) and MSW5W (on West Levee), as well as two sets of wingwalls. Two wingwalls are planned at each abutment at the Gold Street bridge: two at Abutment 1 (south abutment, i.e. West Levee) and two at Abutment 6 (north abutment, i.e. East Levee). Recommendations for floodwalls, MSE walls and wingwalls are included in this report section.

7.2 RECOMMENDATIONS FOR FLOODWALLS

7.2.1 Corps Requirements

We have reviewed (1) our preliminary design parameters in the two letters listed in Appendix D and (2) the CH2MHILL Map and General Plan drawings dated August, 2001. At that time no specific design system was utilized, other than some Caltrans criteria. Subsequent to issuing our draft November 30, 2001 report, we now understand that Corps criteria will be used. Therefore, we have revised our recommendations consistent with the Corps Manual No. EM 1110-2-1913 (Design and Construction of Levees, April 30, 2000) and Corps Manual No. EM 1110-2-2502 (Engineering and Design, Retaining and Flood Walls, September 29, 1989). According to EM 1110-2-1913, levee enlargements "are constructed either by adding additional earth fill or by constructing a flood-wall, "I"-type or "inverted T"-type, on the crown." We subsequently

issued a report dated March 25, 2002 with recommendations consistent with the Corps criteria.

The I floodwall is a vertical wall partially embedded in the levee crown. The stability of such walls depends upon the development of passive resistance from the soil. For stability reasons, I floodwalls rarely exceed 2.13 meters (7 feet) above the ground surface, according to the Corps. One common method of constructing an I floodwall is by combining sheet piling with a concrete cap. The lower part of the wall consists of a row of steel sheet piling that is driven into the levee embankment, and the upper part is a reinforced concrete section capping the steel sheet piling.

An inverted T floodwall is a reinforced concrete wall whose members act as wide cantilever beams in resisting hydrostatic-pressures acting against the wall. According to the Corps, the inverted T floodwall is used to make floodwall levee enlargements when walls higher than 2.13 meters (7 feet) are required.

As described in the 30-Percent Design Submittal, CH2MHILL analyzed the following three wall types.

- Cantilever L-type wall on spread footing (refer to Figure A-2 in the 30-Percent Design Submittal);
- Cantilever inverted T-type wall on spread footing (refer to Figure A-3 in the 30-Percent Design Submittal); and
- Cantilever I-type wall on 600-mm-diameter (24 inches) cast-in-drilled-hole (CIDH) pile with a pile cap (refer to Figure A-4 in the 30-Percent Design Submittal).

Based on their analysis, CH2MHILL recommended that the cantilever L-type wall with spread footing be constructed because of its low initial costs and simpler construction methods. However, we understand the shear key has been moved to coincide with the riverside edge of the footing.

7.2.2 Wall Settlement

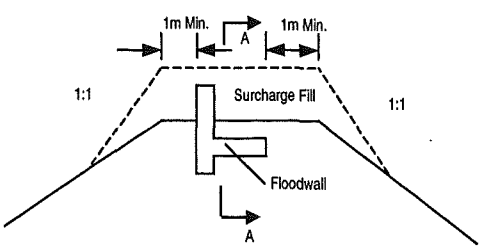
As described in Section 5.3 "Future Settlement", we have estimated future total settlement of the existing levees in Reach G to be less than 13 mm (½ inch). This is the result of ongoing consolidation settlement of the foundation clay layers as a result of the 1963, 1983 and 1995 levee construction and fill placement activities.

We have also estimated additional settlement at FW2E, FW3E, FW4E, FW1W, FW2W, FW3W and FW5W due to new fill to be placed behind the floodwall ranging in height from 0.9 to 1.5 meters (3.0 to 4.9 feet). The estimated additional settlement and rates, including the effect of surcharging each area with 1.5 meters (5 feet) of temporary fill, is shown in the following table. Also included is estimated additional settlement at the Gold Street bridge wingwalls.

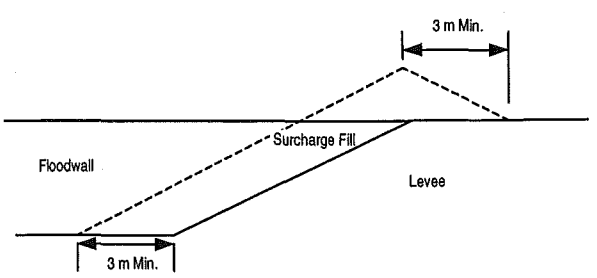
Wall	Approximate Station of Max. Fill Height	Cross Section Used in Analysis	Max. New Fill Height (m)	Estimated Time Rate Settlement (mm)				Ultimate Settlement from New Fill (mm)	90% Consolidation (mm)	Surcharge? ($\rho_{90\%}$ >25mm)	Time to 90% Consolidation (months)	
				1 month	3 months	6 months	1 year				No Surcharge	w 1.5m Surcharge
FW2E	E21+072	#8	0.9	9	13	18	23	30	27	yes	18	4
FW3E	E21+508	#8	1.2	17	25	33	42	54	49	yes	22	3
FW4E	E21+533	#11	1.2	29	47	60	70	76	68	yes	12	1
FW1W	W40+370	#9	1.5	26	37	45	51	53	48	yes	10	1
FW2W	W40+426	#9	1.3	21	30	37	42	45	41	yes	10	1
FW3W	W40+829	#10	1.2	32	48	60	69	76	69	yes	10	1
FW5W	W40+842	#10	1.4	37	56	71	81	88	79	yes	11	1
Abutment 1	W40+406	#9	0.9	17	24	30	33	35	32	yes	12	1
Abutment 6	E21+110	#8	0.4	6	9	12	16	22	20	no	-	-

As shown above, the time to achieve 90% consolidation is about 9 to 22 months. If a surcharge height of 1.5 meters (5 feet) is placed for a period of about 4 months at FW2E and FW3E and about 1or 2 months at FW4E, FW1W, FW2W, FW3W, FW5W and Abutments 1 and 6, 90% consolidation settlement could be achieved. This would result in post-surcharge settlement of less than 6 mm (¼ inch), assuming the walls are constructed after surcharge is removed. If surcharge is selected, we recommend that settlement plates/devices be installed prior to placement of the fill and settlement be monitored before, during and afterwards. These monitoring results should be submitted to and reviewed by URS.

Based on the estimated settlement, we estimate maximum differential settlement of the levee and cantilever L-type floodwall to be approximately 6 mm (¼ inch) in a distance of 30 m (100 feet). The following sketches illustrate the lateral extent of the surcharge fill we recommend.



Typical Floodwall Cross-Section



Floodwall Profile A-A

If this estimated settlement is acceptable to the Structural Engineer and the District, then spread footing foundations would be appropriate. Otherwise, drilled piers are feasible, but should extend below the compressible foundation clay layers. Although the cantilever L-type floodwall will likely be used, recommendations for both foundation types are presented below.

7.2.3 Driving Side Design Earth Pressure

Consistent with the Corps requirements, a tension crack should be assumed to extend vertically down to the bottom of the shear key on the riverside. Therefore, there is no driving side earth pressure to be used for floodwall design. Only hydrostatic pressure need be considered (equal to 9.8 kN/m^3 or 62.4 pcf). This value is recommended, regardless of foundation type.

7.2.4 Resisting Side Design Earth Pressure

7.2.4.1 At Rest Earth Pressure

The estimated moist unit weight of the clay soil adjacent to the floodwall is 20.4kN/m^3 [130 pounds per cubic foot (pcf)]; the corresponding submerged unit weight of the same clay soil is 10.7 kN/m^3 (68 pcf). Assuming that soil will not be excavated adjacent to the wall during its' lifetime, at-rest conditions occur on the resisting (landside) side; for this at-rest condition, a K_O value of 0.66 with an equivalent fluid weight of 13.51 kN/m^3 (86 pcf) acting against the flood walls is recommended. This assumes groundwater is temporarily at the bottom of the footing at the landside edge. When groundwater rises to ground surface above the landside footing edge, an equivalent fluid weight of 16.8 kN/m^3 (107 pcf) is recommended against the flood wall; this is for a combination of at-rest soil and water pressure. The wall resistance to sliding should be evaluated for both loading cases which represent different stages of seepage. The upper 300 mm (1 foot) of embedment should be neglected for design.

We understand most floodwalls will be designed with a shear key. Where there is a shear key or the bottom of slab is inclined, the shear resistance through the clay subgrade should be estimated using (1) a cohesion of 84 kPa (1,750 psf) (undrained condition); or (2) a friction angle of 24 degrees and an effective cohesion of 14 kPa (290 psf). For walls located on the VTA Mitigation Levee, the shear resistance of the clay subgrade should be estimated using (1) a cohesion of 134 kPa (2,800 psf) (undrained condition), or (2) a friction angle of 24 degrees and an effective cohesion of 14 kPa (290 psf), whichever is less.

In the event that a spread footing (horizontal base) with no shear key is selected at a few locations, an ultimate coefficient of friction of 0.325 (tangent of 18 degrees for clay to concrete friction angle) should be used between the base of the retaining wall and underlying soil, with the total resistance not to exceed 36 kPa (750 psf). This assumes (1) that the retaining walls are cast neat against undisturbed engineered fill, (2) the bottom of slab is horizontal and (3) there is no shear key.

7.2.4.2 Passive Pressure

In accordance with the Corps manual for certain loading conditions, resisting-side earth pressure may be increased, but not to exceed one-half the (ultimate) passive pressure using unfactored shear strengths. Again, assuming that soil will not be excavated to the wall during its' lifetime, an ultimate passive resistance of the soil should be estimated using a K_p value of 3.2 with horizontal backfill and an equivalent fluid weight of 65.3 kN/m^3 (416 pcf); for a backfill sloped down at 2:1 we recommend a K_p value of 1.7 with an equivalent fluid weight of 34.7 kN/m^3 (221 pcf). This assumes groundwater is temporarily at the bottom of the footing at the landside edge. When groundwater rises to ground surface above the landside footing edge, an equivalent fluid weight of 34.2 kN/m^3 (218 pcf) for a horizontal backfill ($K_p = 3.2$) and 17.1 kN/m^3 (109 pcf) for a backfill sloped down at 2:1 is recommended against the *flood* wall; this is for an ultimate passive resistance of soil only. Water pressure should be added to this value. The wall resistance to sliding should be evaluated for both loading cases which represent different stages of seepage. The upper 300 mm (1 foot) of embedment should be neglected for design.

7.2.5 Bearing Capacity

The ultimate bearing capacity for spread footings, when supported on new or existing engineered fill material consisting of stiff to very stiff clay soils, is 287 kPa [6,000 pounds per square foot (psf)]. The corresponding ultimate bearing capacity for walls situated on the VTA Mitigation Levee is 575 kPa (12,000 psf). In accordance with Corps EM 1110-2-250, paragraph 4-4, page 4-1, the bearing capacity for loading Cases I1, I2, I3 and I4 should be equal to the ultimate bearing capacity divided by the appropriate factor of safety presented in Table 4-2 of that reference. The corresponding factors of safety listed in Table 4-2 for loading Cases I1, I2 and I4 are 3.0, 2.0 and 2.0, respectively. For Case I3, we recommend using a factor of safety of 1.5 which is consistent with the Corp's requirements; Table 4-2 requires a minimum value greater than 1.0.

It is recommended that the bottom of footing be placed a minimum of 0.91 meter (3 feet) below lowest adjacent finished grade.

7.2.6 Foundation Uplift

A spread footing foundation will be temporarily subject to seepage uplift during the 100-year flood event. We recommend the water pressure on the base be estimated by the line-of-creep method presented in the Corps Manual EM 1110-2-2502. This method is based on estimates of the total heads at the ends of the wall base and then assuming that the total head varies linearly along the shortest possible seepage path along the wall base. As a tension crack is assumed extending the front toe of the wall, the total head will be equal to the water head at the design flood level; then, the uplift pressure is calculated by subtracting elevation head from the total head and multiplying the resulting pressure head by the unit weight of water. The back end of the wall is assumed to have no uplift pressure; then, the uplift pressure along the base will be linearly interpolated.

7.2.7 Drilled Piers

We understand there are no published Corps guidelines or manuals for design of drilled piers. Therefore, in the event drilled piers are selected for Reach G by the Design Team using the Caltrans method (Memo to Designers 22-1, dated July 2001), we recommend the geotechnical design parameters in the following table.

Geotechnical Parameter	Recommended Value
Design Unit Skin Friction for Piles	24 kPa (500 psf)
Isolation Factor	1.0
Ka (active coefficient)	0.36
Kp (passive coefficient) 2H:1V Slope Downward	1.7
Kp (passive coefficient) Horizontal	3.2

For pile embedment length less than 12 times the diameter, it can be assumed that the pile is relatively stiff and the depth of embedment can be determined by structural analysis consistent with the procedure outlined in the U.S.S. Steel Piling Design Manual (July 1975).

7.2.8 Lateral Load Capacity of Drilled Piers

The drilled pier foundation alternative will be capable of resisting lateral loads. The magnitude of the lateral load resistance is dependent on many factors, including the pile size and embedment length, condition of pile cap fixity, the physical properties of the surrounding materials, and the yield moment capacity of the pile. To estimate the lateral load capacity of an individual drilled pier, we recommend using methods that are based on the p-y analysis. Commerically available computer program such as LPILE 4.0 (Reese et al, October 2000) is a widely acceptable tool for this type of analysis. We understand that CH2MHILL is planning to use this program for the analysis of lateral load capacity of the drilled pier foundation alternative. The following table summaries our recommended geotechnical input parameters for the LPILE program:

Layer	LPILE Layer Type	Effective Unit Weight (kN/m^3)	Soil Modulus Parameter, k (kN/m^3)	Soil Shear Strength (kPa)	Strain at 50% Maximum Stress
1	Stiff Clay with Free Water	10.8	135,700	96	0.005
2	Stiff Clay with Free Water	11.6	108,600	86	0.007
3	Stiff Clay with Free Water	9.7	67,800	57	0.01

Based on an example LPILE run, we estimate the pile head deflection and maximum moment to be 13 mm (0.5 inch) and 390 m-kN (290 ft-kip), respectively, at a depth of

approximately 2.1 meters (7 feet). These estimates are based on a shear of 312 kN (70 kips) and zero moment at the pile head, and the boundary condition of the pile head is assumed to be "free." Based on the proposed bottom of the wing wall (Elev. 2), we assume level ground surrounding the pile.

We recommend a group reduction factor of 0.7 for pile caps having two or more rows of piles. These recommended group reduction factors are based on the assumption that the center-to-center spacing will be at least 3 times the pile diameter.

7.2.9 Construction Considerations for Drilled Piers

The conditions encountered in the borings consist primarily of layers of lean clay and fat clay. Groundwater was encountered at about Elevation –1 to –2 meters, which corresponds to the approximate existing channel bottom. The August 2001 drawings also show a mean daily high tide at about Elevation 1.9 to 2.1 meters in this area. Therefore, during construction we would expect groundwater as high as Elevation 2.1 or higher depending on rainfall. While the cohesive deposits will help to keep the hole open during construction, some sloughing and caving can be expected if cohesionless deposits are encountered. Therefore, it is recommended that the Contractor have casing on site, should it be needed to minimize the risk of caving in either clay or sand soils. If casing is used, the casing should be withdrawn from the hole slowly as the concrete is being placed; a minimum head of concrete of 0.9 meter (3.0 feet) should be maintained in the casing at all times. All CIDH piles should be installed under the direct observation of a representative of the Geotechnical Engineer.

7.3 RECOMMENDATIONS FOR WINGWALLS

7.3.1 General

According to sheets CD-11 and CD-12 of the July 2002 Design Drawings, wingwalls for the Gold Street bridge are planned at Abutment 1 (south abutment) and Abutment 6 (north abutment), respectively. Two wingwalls are shown at each abutment. We understand Caltrans Type 1 retaining walls are planned. Wall lengths range from 4.500 to 7.450 meters (14.76 to 24.44 feet) and wall heights vary from 4.285 to 4.525 (14.05 to 14.84 feet). Bottom of pile cap at all four wingwalls is positioned at Elevation 2.000 meters (6.56 feet).

Our recommendations herein pertain to the wingwalls proposed at the intersection of Gold Street and Guadalupe River (both sides of the river and both sides of the roadway).

7.3.2 Lateral Earth Pressure

The moist unit weight of the soil near (and behind) the retaining walls is estimated to be 18.1 kN/m^3 (115 pcf). We recommend the following lateral earth pressures (for both wet and dry conditions).

- For horizontal slope conditions behind and in front of the wall, $K_A = 0.33$, $K_O = 0.5$ and $K_P = 3.0$

- For 2:1 (horizontal to vertical) positive slope conditions behind the wall and negative in front of the wall, $K_A = 0.55$, $K_O = 0.6$ and $K_P = 1.1$
- For 1.5:1 positive slope conditions behind the wall and negative in front of the wall, $K_A = K_O = K_P = 0.75$

These values are based on a frictionless vertical retaining wall that is in contact with the soil.

In addition we recommend the following soil strength parameters.

- Drained case – Angle of Internal Friction (ϕ) = 30 degrees and cohesion (c) = o
- Undrained case – ϕ = 0 degrees and c = 25 kPa (500 psf)

7.3.3 Deep Foundations

For these wingwalls, we anticipate deep foundations will be needed. Therefore, the following ultimate adhesion (skin friction) values can be used for driven piles:

- Native soils above Elevation (-10 m), adhesion = 25 kPa (500 psf)
- Native soils below Elevation (-10 m), adhesion = 50 kPa (1,000 psf)

It should be understood that fill depths measured in borings drilled in the vicinity of Gold Street generally extend to about Elevation 0. In addition we recommend a safety factor (F.S.) of 2.0 for an allowable adhesion value.

7.4 MSE WALL DESIGN

7.4.1 Wall Configuration

Based on the July 2002 Design Drawings and subsequent information provided by CH2MHILL, we understand that two Mechanically Stabilized Embankment (MSE) walls are planned *on* the outboard *side* of the levees. The reason for these MSE walls is the limited District right-of-ways. The planned configuration of these walls was previously described in Section 1.1 "Project Description". Typical MSE wall orientation as well as typical loading conditions are shown in Figure 51.

7.4.2 Materials

The embankments behind the MSE walls are planned to be constructed either horizontally or at 2 to 1 (horizontal to vertical) slopes. The MSE walls should be constructed with compacted, relatively clean, granular imported material meeting the requirements of Caltrans Standard Specifications Section 19-3 and the following criteria.

- Particle size ≤ 150 mm
- Amount passing the 76 mm Sieve, 78 to 100%
- Amount passing the 0.074 mm (No. 200 Sieve) ≤ 25%

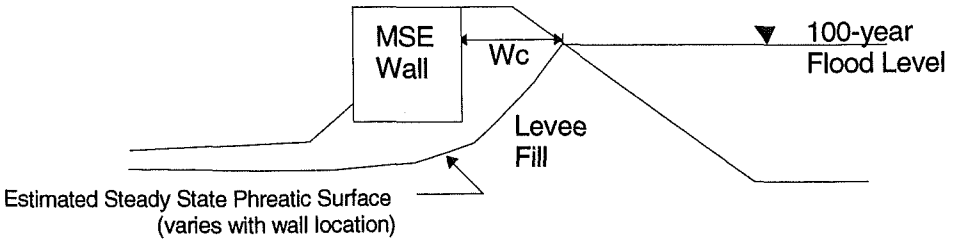
- Plasticity Index < 10
- Angle of Internal Friction, $\phi \geq 34$ degrees
- Sand equivalent ≥ 20

7.4.3 Seepage Considerations

Since an MSE wall typically consists of granular backfill reinforced with metallic strips or wire mesh, the wall unit is expected to have a higher permeability than the adjacent levee fill materials. To evaluate any adverse effect on the seepage control characteristics of the levees, we elected to establish an effective minimum section that a levee should have under the design 100-year flood event and then compare that to the planned MSE wall configurations. This effective minimum section is based on the following guidelines:

- A minimum crown width of 3.05 to 3.66 meters (10 to 12 feet) as outlined in the Corps' Manual for "Design and Construction of Levees", EM 1110-2-1913, Chaper 6.
- A minimum impervious core width of 3.3 meters (10 feet) for general dam/levee safety design.

Based on the above guidelines and our experience, we recommend a practical minimum section of 3.3 meters (10 feet) of impervious clay levee fill be required for these levees under the design 100-year flood event. Please see the following sketch illustrating the measurement of the effective minimum section (Wc).



In addition, we have estimated the seepage paths using SEEP/W finite element seepage analysis for the various levee cross-sections. We have assumed steady state seepage using 100-year flood level. Based on the seepage analyses, we compared the resultant phreatic surface with the proposed configuration of the MSE walls.

Our evaluation indicates that both proposed MSE walls meet the required minimum width Wc of 3.3 meters (10 feet). Our analysis also indicates the phreatic surface will fall below the bottom of both MSE walls; therefore, it is unlikely that seepage from the river (100-year flood event) will intercept walls MSW1E and MSW5W. Nevertheless, seepage through the levee and then through these two MSE walls should be anticipated in design. We estimate the seepage quantity flowing into the MSE granular backfill will be relatively small (below 1/10 gallon per minute per lineal foot of wall) and the gradient at the exit toe to be less than 0.5. Since these walls consist of granular backfill, we expect the small quantity of floodwater, if any, that might seep through the backfill and eventually exit the walls through the front panel joints would result in little to no buildup

of hydrostatic pressure in the walls. We have assumed that an envelope of permeable material will be placed between the granular backfill and the precast concrete wall panels, and geotextile filter fabric will be placed between the permeable material backfill and concrete wall panels.

In addition, we recommend that new fill placed above the MSE wall (mostly where there is a sloping backfill) consist of relatively impervious lean clay. If possible, this compacted clay layer should be at least 1 meter (3.3 feet) thick. The purpose of this clay cap is to minimize rainfall infiltration behind the precast concrete wall panels.

Since these walls meet the minimum levee section required and the potential seepage quantity is low, if any, we expect the MSE wall to have no adverse effect on the seepage control characteristics of these levees.

7.4.4 Driving Side Lateral Pressures

The MSE walls should be designed to resist lateral earth pressures imposed by the adjacent backfill. It is recommended that the walls be designed to resist a lateral pressure based on a coefficient (K_a) of 0.32. Note that for walls with sloping backfill, the pressure diagram behind the reinforced fill section should extend to the top of levee or slope (see Figure 51). Surcharge loads such as vehicular traffic would also apply additional loads to the walls. For design purposes, a surcharge pressure equivalent of 0.6 meter (2 feet) of soil should be applied to account for traffic loads. A surcharge unit weight of 20 kN/m^3 can be assumed for design.

The recommended pressures assume that no hydrostatic pressures will develop behind the MSE walls. The buildup of hydrostatic pressure behind the walls should be prevented by the inclusion of a positive drainage system. It is recommended that such a system consist of a series of permeable material wedges placed behind the precast concrete wall panels.

7.4.5 MSE Stability

The following discussion presents geotechnical parameters for the Structural Engineer to evaluate the stability of MSE walls. For internal stability of the wall elements, the Structural Engineer's design should be consistent with the wall manufacturer's published guidelines and/or Caltrans design procedures.

7.4.5.1 Sliding and Overturning Stability

It is recommended that the sliding and overturning stability of the MSE walls be evaluated during design using the following parameters:

Engineering Property	Granular MSE Backfill	Foundation Soil	
		MSW5W	MSW1E
Friction Angle, ϕ (degrees)	34	31	31
Cohesion, C (kPa)	0	48	70
Total density, γ (kN/m^3)	19	16	16

Refer to Figure 51 for the loading diagram which presents the sliding forces and overturning moments. For sliding stability, a friction coefficient of 0.48 (ultimate) should be assumed to exist at the base of the wall to resist sliding. The friction coefficient is based on the estimated friction resistance between the granular backfill and reinforcement, i.e. 0.75x34 degrees = 25.5 degrees.

We recommend that the walls be designed to achieve minimum stability safety factors of 1.5 for static and 1.1 for seismic conditions for the sliding mode of failure. For overturning stability, the minimum factor of safety should exceed 2.

7.4.5.2 Global Stability

Global stability is defined as the evaluation of a deep seated failure, i.e. below and outside the MSE wall elements. The global stability of the MSE walls was evaluated using a SLOPE/W (Geo-Slope, Int., 1992). In Spencer's procedure, all forces are assumed to have the same inclination, and all requirements for static equilibrium are satisfied. The trial and error solution involves successive assumptions for the factor of safety and side force inclination until both force and moment equilibrium are satisfied.

The global stability of the two retaining walls was analyzed for the end of construction condition. The end of construction case represents the condition immediately following construction when excess pore pressures have been induced by the construction, and not enough time has passed for them to dissipate. For this reason, undrained conditions exist for impervious clay soils, whereas drained strengths were used for granular soils. Strength results from laboratory unconfined compressive strength tests were used for the fine grained soils and the estimated friction angle was used for the granular soils. In our engineering opinion, the end of construction represents the most critical loading condition.

The minimum required factor of safety for global stability is 1.3. The minimum (critical) factors of safety computed are summarized in the table below.

Retaining Wall No.	Minimum Calculated Factory of Safety
MSW5W	9.0
MSW1E	5.0

The calculated factors of safety all exceed the minimum required value of 1.3.

7.4.6 Bearing Capacity

MSE walls designed in accordance with the previous recommendations should be designed for ultimate bearing capacity as follows:

Wall No.	Ultimate Bearing Capacity (kPa)
MSW5W	370
MSW1E	395

We recommend that the walls be designed to achieve a bearing pressure less than one half the ultimate bearing capacity.

7.4.7 Wall Settlement

As described in Section 5.3 "Future Settlement", we have estimated future total settlement of the existing levees in Reach G to be less than 13 mm (½ inch). This is the result of ongoing consolidation settlement of the foundation clay layers as a result of the 1963, 1983 and 1995 levee construction and fill placement activities.

We have also estimated additional settlement at MSW5W and MSW1E due to the localized fill as high as 1.3 meters (4.3 feet) to be placed for maintenance ramps, as shown in the following table.

Wall	Station	Cross Section Used in Analysis	Max. New Fill Height (m)	Estimated Time Rate of Settlement (mm)				Ultimate Settlement from New Fill (mm)	90% Consolidation (mm)
				1 month	3 months	6 months	1 year		
MSW5W	W40+500	#10	1.3	35	53	67	76	83	75
MSW1E	E21+525	#11	1.3	25	36	46	53	57	52

As shown above, it is estimated that 83 mm (3¼ inches) of settlement will occur at MSW5W, and 57 mm (2¼ inches) of settlement will occur at MSW1E.

7.4.8 Leveling Pad

In order to provide a uniform bearing pad and sliding resistance (along the soil block-foundation interface), we recommend soil be subexcavated to a depth of 0.3 (1 foot) meter below the bottom of the leveling pad and the subexcavation be backfilled with the same backfill material listed in Section 7.4.2 "Materials" ($\phi \geq 34°$) and used within the reinforced-soil block. The subexcavation area should extend laterally at least 0.3 meter (1.0 foot) on both sides of the leveling pad.

7.4.9 Earthquake Considerations

Each MSE wall structure consists of horizontal wire mesh/strips placed between granular layers. In cross section, it is rectangle filled with granular soil placed within and next to the clay levee. These materials will perform differently during an earthquake. We believe that some cracking will occur at or adjacent to the interface between the MSE sand and the lean clay. Consequently, the District should be prepared to repair these cracks after major earthquakes.

7.5 CORROSION INVESTIGATION

As assessment of the potential for corrosion of various buried foundations and structures was performed by Villalobos & Associates. The results of their investigation are presented in Appendix E.

Based on the soil corrosivity data and review of the project requirements, corrosion monitoring and control are recommended to minimize corrosion of metallic and reinforced structures. These recommendations are in accordance with Caltrans Bridge Design Specifications, Section 8.22. The quality of materials and workmanship should be inspected during construction by a corrosion engineer.

Prestressed Concrete Piles

Prestressed concrete piles should be constructed of durable concrete such as described in ACI Standards 201.2R and 222R. These recommendations include but are not limited to the following:

- The water/cement ratio should not exceed 0.40.
- A concrete cover of a minimum of 2 inches should be applied over all steel reinforcement.
- Mineral admixtures conforming to ASTM Designation C1240 and/or ASTM Designation C618 Type F and/or N should be used.
- Salt-free sand and potable water should be used.
- The concrete should be allowed to cure according to the manufacturer's recommendations.
- Mineral admixtures conforming to ASTM Designation C1240 and/or ASTM Designation C618 Type F and/or N should be used.

Buried Metallic Structures

Based on the low soil resistivity it is recommended that buried metallic piping systems, such as steel and ductile iron, be coated with a bonded dielectric or mortar coating. All non-welded joints should be bonded to provide electrical continuity. Cathodic protection should be installed to provide corrosion protection. Buried metallic piping should be electrically isolated from other structures, or where a change in piping or coating material occurs.

Buried Reinforced Concrete Structures

Buried reinforced concrete structures should be constructed of durable concrete such as described in ACI Standards 201.2R and 222R. These recommendations include but are not limited to the following:

- The water/cement ratio should not exceed 0.40.
- A concrete cover of a minimum of 3 inches should be applied over all steel reinforcement.

- Salt-free sand and potable water should be used.
- The concrete should be allowed to cure according to the manufacturer's recommendations.

MSE Retaining Walls

For permanent structures, the reinforcement elements shall be designed to have a minimum design life of 75 years. For the metallic reinforcement, galvanization loss will be:

- 15 μm/year/side for the first 2 years
- 4 μm/year/side for subsequent years

The loss of bare steel after the galvanized zinc is depleted will be 12 μm/year/side. These estimates are from the AASHTO Standard Specifications for Highway Bridges, Section 5.8.6.1. It is assumed that the soil backfill used is nonaggressive (i.e. Class 1 Backfill), meeting the following criteria:

- pH of 5 to 10
- Resistivity of not less than 3,000 ohm-cm
- Chlorides not greater than 100 ppm
- Sulfates not greater than 200 ppm.

The above mentioned thicknesses are not valid if the following conditions are present:

- The MSE wall is exposed to marine or other chloride rich environments
- The MSE wall will be exposed to stray currents such as from nearby underground power lines or adjacent railways
- The backfill material is aggressive

SECTION 8

Soil liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to the cyclic loading associated with earthquake ground shaking. Based on the latest ground motions anticipated for this site (Section 2 "Site Geology and Seismicity"), we have updated the evaluation of liquefaction potential performed previously for our April 2000 study. The current evaluation was based on semi-empirical methods considering the following three fault systems: 1) a moment magnitude 8.0 earthquake on the San Andreas fault, 2) a moment magnitude 7.1 on the Hayward fault and 3) a moment magnitude 6.5 on the Hayward Southeast Extension fault. The basis of these earthquake magnitudes and site accelerations is described in the Section 2, "Site Geology and Seismicity."

The clay layers at the site are not susceptible to liquefaction. Therefore, the potential for liquefaction of various sand and gravel layers at the site was evaluated. On the basis of our analysis, it appears that there is some potential for liquefaction to occur at the locations shown on Figure 2, Location of Potentially Liquefiable Layers. The potentially liquefiable layers appear to be concentrated in the region of the levee alignment near the UPRR tracks, although a few scattered potentially liquefiable layers exist along the remainder of this reach of the levee alignment. These layers range in thickness from 0.6 to 4.0 meters (2 to 13 feet). Should liquefaction occur, settlement could occur at these locations. Settlement of the ground surface is estimated to be on the order of 12 to 110 mm (½ to 4½ inches) at these locations. Estimates of these potential settlements were made using the procedures outlined by Tokimatsu and Seed, (1984).

Based on explorations conducted, most layers of potentially liquefiable soil were encountered below depths of at least 3 meters (10 feet) below ground surface at landside toe. These deposits are overlain by layers of non-liquefiable cohesive soils. Empirical research by Ishihara (1985) indicates that for level ground, liquefaction is not likely to result in visible damage at the ground surface as long as the cover of the liquefiable soil is equal to at least 3 meters (10 feet). Based on Ishihara's research and our local experience, we expect ground surface rupture is unlikely.

As discussed in our April 2000 report, we conclude that lateral spreading is unlikely at the liquefaction prone locations in this reach.

We performed post-liquefaction slope stability analysis at one critical cross-section (Cross-Section 11) using residual strengths estimated for the potentially liquefiable sand layers. The factor of safety against undrained loading simulating conditions immediately after liquefaction is above 1.3. Therefore, we conclude that lateral spreading is unlikely at the liquefaction prone locations in these reaches. The results of the above analysis are presented below.

Cross Section			Factor of Safety End of Construction			Residual Undrained Shear Strength
Number	Station	Levee	Minimum Required	Riverside	Landside	Psf
11	7+800	East	1.3	2.2	3.0	600

SECTION 9

9.1 GENERAL

All site preparation and earthwork should be done under the observation of a representative of the Geotechnical Engineer and in accordance with the recommendations presented herein. In general, grading/earthwork will include raising crown and side slopes of existing levees, constructing a new levee, excavating and backfilling for new floodwalls, excavating for new tieback walls, constructing depressed maintenance roads and ramps and removal of sediment in the channel.

9.2 CLEARING AND DEMOLITION

All concrete flatwork, existing pavements, aggregate base, armoring aggregate, loose stone, drainage lines, underground utilities, trees, brush, vegetation, fencing and abandoned structures designated for removal should be demolished and removed from the site. Abandoned utilities encountered during grading should be removed in their entirety. Existing Geotextile Soil Reinforcing (GSR) walls are located on the crown of the following levees: the East Levee north of Route 237, on the West Levee between Gold Street and Route 237, on the East Levee north and south of Tasman Drive and on the West Levee north of Tasman Drive. According to the 1995 As-built drawings, semi-impervious or structural backfill ($PI \leq 25 \pm 5$) was placed inside the outermost 1.5 to 2.7 meters (5 to 9 feet) landside portion of the GSR. If left in place, there is the potential for horizontal seepage, particularly through the semi-impervious material. Therefore, these GSR walls should be removed in their entirety.

9.3 GRUBBING

Grubbing consists of the removal, within the levee foundations, floodwalls, maintenance road and ramp areas, of all stumps, roots, buried logs, old piling, old paving, drains, and other objectional matter. Roots or other intrusions over 38 mm (1½ inches) in diameter within the levee foundations, floodwalls and maintenance road areas should be removed to a depth of 0.91 m (3 feet) below natural ground surface. Shallow tile drains sometimes found in agricultural areas should be removed from the levee foundation areas. Materials resulting from grubbing operations should be removed from the site. The sides of all holes and depressions caused by grubbing operations should be flattened before backfilling. Backfill consisting of material similar to adjoining soils, should be placed in layers up to the final foundation grade.

9.4 STRIPPING

After foundation clearing and grubbing operations are complete, stripping should be commenced. The purpose of stripping is to remove low growing vegetation and organic topsoil, including but not limited to the side slopes and crown of existing levees, the landside and waterside toe of existing levees, and the footprint of new levees and maintenance roads and ramps. The depth of stripping should be determined by local conditions and is expected normally to vary from 152 to 305 mm (6 to 12 inches). The final depth of stripping should be determined by a representative of the Geotechnical Engineer during site preparation. Materials resulting from stripping operations should be

removed from the site. Organic topsoil that does not contain debris may be stockpiled, if desired, for re-use as topsoil in landscape areas if approved by the Landscape Architect. The stripped materials should not be re-used as engineered fill or blended with other materials.

9.5 REMOVAL OF SEDIMENT IN CHANNEL

The channel sediment areas should be excavated as shown on the drawings to bring those areas to their finish channel elevations. All sediment materials should be removed from the site and should not be re-used for construction of levees, maintenance roads, ramps and backfill for floodwalls. Organic topsoil that does not contain debris or contaminants may be stockpiled, if desired, for re-use as topsoil in landscape areas if approved by the Landscape Architect.

9.6 EXCAVATION

After the areas to receive fill have been properly cleared, grubbed and stripped, it is recommended that existing loose fills or soft deposits be removed to the underlying native soils and replaced with engineered fill. Floodwall and tieback wall areas should be excavated as required to bring those areas to their finish subgrade elevations. All loose soil should be removed from the exposed floodwalls and tieback walls subgrade prior to wall construction.

9.7 SUBGRADE PREPARATION

After demolition, stripping and subgrade excavations are complete, the exposed surface to be filled should further be prepared by scarifying the top 152 mm (6 inches) of the surface, moisture conditioning the material to near the optimum moisture content, and recompacting to a minimum relative compaction of 90 percent of its maximum dry density in accordance with ASTM Test Designation D 1557. This procedure is intended to provide a tight bond between the foundation and fill, and to eliminate a plane of weakness at the interface. The foundation surface should be kept drained and not scarified until just prior to fill placement in order to minimize the risk of saturation from rainfall. The base of the exploration trench (just north of Highway 101) should also be scarified, moisture conditioned and compacted prior to the placement of new fill materials. The fill soils exposed in footing excavations for floodwalls and tieback walls are not expected to require scarification. However, the final need for scarification should be determined at the time of excavation.

At some locations, such as at toe of existing levees or depressed maintenance roads, the subgrade surfaces will be positioned near or below the free groundwater level or in areas which have been subjected to standing water. The exposed materials are expected to consist of low plasticity clays, silts and fine sands.

Control of the water in most shallow excavations in these materials probably could be accomplished by sloping them to drain and pumping from sumps within the excavated areas. After dewatering, some of the subgrade surfaces might be stable enough to support

construction equipment; whereas, other areas might develop pumping or rutting. Please refer to Section 9.10 "Construction Dewatering", for further discussion regarding this subject.

In order to minimize the potential for pumping subgrade conditions, it is recommended that the first lift of fill be "track-walked" in place to a thickness of about 0.3 meter (1 foot) using a bulldozer or other track-mounted equipment. The initial lift should be dry of the laboratory optimum and compacted to a minimum relative compaction of 90 percent. Rubber tired equipment, such as scrapers or bottom dump trucks, should not be permitted to traverse the area. Pumping or rutting will be further exacerbated by repeated passes of rubber tired equipment. This could occur wherever a silt or clay subgrade is excavated in the existing levee fill or native soil.

In the event that deep seated pumping develops, it is recommended that the unstable subgrade areas be "bridged" using a combination of stabilization fabric (Mirafi 600X or equivalent) covered by a layer of granular, bridging material. The bridging material should consist of "bank run" or "crusher run" material conforming to the gradation discussed in Section 9.8.3.2 "Bridging Material." Since bridging material is highly permeable, seepage should be expected in this layer. Therefore, bridging material should only be used in areas where seepage is acceptable.

After the reinforcing fabric has been placed on the exposed subgrade surface, the bridging material should be track-walked into place over the fabric. It is estimated that a 0.46 to 0.61 meter (18 to 24 inch) thick layer of bridging material probably will be needed. Rubber tired equipment should not be permitted to traverse pumping areas until placement of the bridging material and stabilization fabric have been completed. Then additional fill can be placed and compacted to provide desired finished grades.

In areas where silt or clay subgrade pumps or ruts (in levee fill or native soil), lime treatment is an alternative treatment to achieve stabilization. Since lime stabilization results in a relatively impervious soil, it can be used anywhere in the levee cross section. Where subgrade areas are silty or clayey sand, they may require a combination of quicklime and flyash stabilization. The Contractor should retain his own testing laboratory to perform tests to determine the appropriate lime and/or flyash content; this should include unconfined compressive strength tests. It has been our experience on other local projects that 4 to 5 percent quicklime by weight will generally achieve the desired stabilization results. All lime/flyash stabilized subgrade should be prepared in accordance with Section 24 "Lime Stabilization" of the July 1992 Caltrans Standard Specifications. The Contractor should retain a Specialty Subcontractor experienced with lime stabilization. Furthermore, the Contractor should include this work in his bid price.

9.8 FILL MATERIALS

9.8.1 Approval

Soil materials, whether from sources on or off site, should be approved by a representative of the Geotechnical Engineer for the intended use and specifically for a

required location or purpose. It is expected that most materials will be imported.

9.8.2 Onsite Materials

9.8.2.1 Channel Sediment

Where it is planned to remove portions of the channel (sediment), these materials should not be re-used as engineered fill. Organic topsoil that does not contain debris or contaminants may be stockpiled, if desired, for re-use as topsoil in landscape areas if approved by the Landscape Architect. Otherwise, these materials should be removed from the site.

9.8.2.2 Excavation for Floodwalls, MSE Walls, Wingwalls, and Tieback Walls

The existing levee is engineered fill composed primarily of stiff to hard compacted lean clay with occasional fat clay. Where is it is planned to remove portions of the existing levee for new floodwall foundations and new tieback walls, MSE walls, and wingwalls, the materials could be reused as new levee fill providing they are properly processed to achieve a moisture content near the optimum prior to placement. The drying of materials with excessive moisture content should be in accordance with the requirements in Section 9.9 "Fill Placement and Compaction". However, excavation for the tieback walls, MSE walls, and wingwalls will require some selectivity to be exercised if these materials are to be considered for re-use as possible new levee fill. Where excavations for tieback walls, MSE walls, and wingwalls are made into existing clay fill, these materials may be re-used; whereas if these excavations extend deeper into organic soils or other objectionable material, these materials should not be re-used as engineered fill.

9.8.2.3 Geotextile Soil Reinforcing Walls

Where Geotextile Soil Reinforcing (GSR) walls are encountered, these walls should be removed in their entirety, to eliminate the potential for horizontal seepage. These materials should be hauled offsite.

9.8.2.4 Existing Armoring Aggregate

The existing surficial layer of armoring aggregate (on the crown, i.e. crest), should be removed. If left-in-place, this horizontal layer of aggregate material could allow seepage through and onto the landside slope. This material could be re-used for surfacing new maintenance roads, provided it is not contaminated or mixed with fine-grained clay or silt soils.

9.8.3 Import Materials

9.8.3.1 General Levee Fill

It is recommended import materials used for levee construction consist of a soil or soil-rock mixture free from organic matter or other deleterious substances. The material should contain at least 50 percent by weight of clayey soils (passing the U.S. Standard No. 200 sieve). The clayey soils should be of low to medium plasticity as classified by

the Unified Soil Classification System having a plasticity index within the range of 10 to 20 and a liquid limit less than 40. The material should not contain rocks or lumps over 152 mm (6 inches) in greatest dimension and should not contain more than 15 percent greater than 64 mm (2½ inches). Sand and gravel size particles should comprise no more than 50 percent of the soil-rock mixture. It is anticipated that materials meeting these general requirements are available from local quarries or borrow pits located offsite.

9.8.3.2 Bridging Material

Bridging material, for bridging soft subgrades for maintenance road areas, should consist of a reasonably well graded mixture of coarse angular gravel and cobbles. The following gradation limits are recommended as a general design guideline.

Sieve Size		Percentage Passing
(mm)	(inches)	
152	6	100
51	2	0 – 50
19	¾	0 – 10
Durability Index: 25 minimum		

9.8.3.3 Armoring Aggregate

Armoring aggregate for placement on the crown of the levees, maintenance roads and ramps, should consist of clean, hard and durable gravel or crushed rock, conforming to the following gradation.

Sieve Size		Percentage Passing
(mm)	(inches)	
76	3	100
38	1½	75-55
19	¾	60-40
13	½	45-30
10	3/8	35-20
6	¼	25-15
No. 200	No. 200	20-0

The rock for armoring aggregate should have a minimum specific gravity of 2.40 in accordance with California Test Method No. 206 and a minimum durability index of 35 in accordance with California Test Method No. 229.

9.9 FILL PLACEMENT AND COMPACTION

All levee fills should be spread in uniform horizontal lifts not exceeding 203 mm (8 inches) in uncompacted thickness. Before compaction begins, the fill should be brought to a moisture content that will permit proper compaction by either aerating the material if it is too wet, or spraying the material with water if it is too dry. The lean clay or occasional fat clay should be compacted in the range from optimum to 3 percent wet of laboratory optimum. The native granular materials should be compacted near the laboratory optimum. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of water content. To prevent drying of the subgrade soils, placement

of fill should start immediately after surface preparation and should proceed in a continuous operation until the site is brought to grade. No fill should be placed during rain or when saturation will hinder proper compaction. The finished grade surface of the compacted fill under floodwall, MSE wall, wingwall, and tieback wall footings should be kept in a moist condition prior to the placement of concrete. Jetting or flooding of the fill should not be permitted.

Relative compaction is the ratio of the in place dry density of the constructed fill to a maximum dry density determined by ASTM Test Designation D 1557. All levee fills should be compacted to a minimum relative compaction of 90 percent, as determined by ASTM Test Designation D 1557. All pavement aggregate base course, subbase and subgrade materials, and any fill material which is within the upper 600 mm (2 feet) below finished pavement surface should be compacted to a minimum relative compaction of 95 percent. All fill material below the bottom of footings and all other structural elements should be compacted to a minimum relative compaction of 95 percent. All other fill materials should be compacted to a minimum relative compaction of 90 percent. A representative of the Geotechnical Engineer should be present to observe all grading operations during both preparation of the site and compaction of engineered fill.

The compaction and moisture content of the subgrade and each layer of levee placed should be tested by a representative of the Geotechnical Engineer in accordance with ASTM D 1556 "Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method" or ASTM D 2922 "Standard Test Method for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)", ASTM D 3017 "Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth)" and ASTM D 2216 "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock."

Fills placed on slopes steeper than 6:1 (horizontal to vertical) should be benched into the adjacent soils to provide a bond at the interface. Loose soils on the surface of existing slopes should be excavated as new fill is placed. Benching is expected to be necessary where new fill will be placed on existing levee slopes in order to raise the existing levee. It is recommended that horizontal benches be constructed such that the maximum height of the vertical section of the bench does not exceed 305 mm (1 foot).

Where new fill is placed on the face of an existing levee, the fill should extend laterally a minimum of 0.61 meter (2 feet) beyond the finished lines during construction. After compaction, the excess fill should be removed to the final grade and lines.

9.10 CONSTRUCTION DEWATERING

Because the groundwater level is below the bottom of the anticipated depth of excavations for floodwalls, MSE walls, wingwalls, tieback walls, and the "inspection" or "exploration" trench for levee just north of Highway 101, the excavation bottoms are expected to be relatively dry during the excavation phase. In the event that water

becomes ponded during the rainy season, the ponded water could be removed by sump pit/pump procedures.

Depending on the season of the year, the water level could vary considerably in the Guadalupe River channel. Although elevations of the riverside maintenance roads have not yet been established, it is conceivable that some road subgrade areas may be soft and wet. As discussed previously, those areas should be over-excavated and bridging material placed to a maximum depth of 0.61 meter (2 feet); alternatively, lime stabilization treatment could be used. The water encountered could be removed by sump pit/pump procedures.

SECTION 10

10.1 GENERAL

Based on our site reconnaissance, as well as our subsurface investigation and geotechnical analyses, we believe the geotechnical condition of the existing East and West Levees between UPRR and Route 237 as described herein is generally very good. These opinions and our recommendations are discussed below in more detail in this section.

10.2 RESULTS OF SLOPE STABILITY EVALUATION

10.2.1 Levee Slopes

In Section 4 "Levee Slope Stability Evaluation", we discussed (1) the cross sections selected for analysis, (2) method of analysis and (3) stability results. We identified a total of six (6) levee sections as candidates for analysis. Based on our slope stability analyses of these six sections, we conclude that all of these sections have factors of safety that met or exceeded the minimum factor of safety required for four (4) Corps' design conditions. However, based on earthquake loading conditions, we anticipate permanent deformation of the levees to be in the range of 80 to 110 mm (3 to 4 inches), except at Section 12, where deformation is estimated to be nil.

As previously stated, it is unlikely that the design earthquake event will coincide with a period of high water. However, all levees should be inspected following a major seismic event since deformations due to earthquake shaking are anticipated. Any levee repairs and/or upgrades should be consistent with the District's levee safety program.

10.2.2 Depressed Maintenance Road Slopes

In addition, we also discussed in Section 4.4 "Stability Results of Depressed Maintenance Road" planned south of Gold Street at the riverside toe of both levees. Based on our slope stability analyses of these maintenance road sections, we conclude that these sections have factors of safety that exceeded the minimum factor of safety required by the Corps for end of construction condition.

10.3 RESULTS OF SETTLEMENT ANALYSES

In Section 5 "Settlement Evaluation", we discussed (1) land subsidence, (2) inferred levee settlement, (3) predicted settlement and (4) future levee settlement. The latter is based on consolidation of the foundation clay soil due to previous filling in 1963, 1983, 1995 and 1999. On that basis, we estimate the future long term consolidation settlement to be less than 13 mm (1/2 inch) for the West, East and Mitigation Levees.

In Section 7.2.2 "Wall Settlement", the estimated time to achieve 90% consolidation for floodwalls is about 9 to 22 months. If it is desired to construct the floodwalls before the 9 month period, then a surcharge fill with a minimum height of 1.5 meters (5 feet) is recommended. The surcharge settlement periods are listed in detail in Section 7.2.2.

We estimate maximum differential settlement of the levee and floodwall (placed on spread footings) to be about 6 mm (1/4 inch) in a distance of 30 meters (100 feet). If this estimated settlement is acceptable to the Structural Engineer and the District, then spread footing foundations are recommended. The floodwall should be overbuilt to accommodate this future settlement; the Mitigation Levee should also be overbuilt to accommodate this future settlement. If these settlements are not acceptable to the Structural Engineer and the District, then drilled piers are recommended for the support of the floodwalls. Drilled piers should extend below the compressible foundation clay layers, however. In addition, a concrete cutoff wall should extend a minimum of 1 meter (3.3 feet) below lowest adjacent finished grade. Since the levee will settle relative to the floodwall, periodic grouting may be required to fill any voids between the wall foundation and levee fill soil.

At MSE wall locations MSW1E and MSW5W, we estimate maximum future settlement of about 13 mm (1/2 inch) to occur due to previous filling in 1963, 1983, 1995 and 1999. Due to new fill to be placed for new maintenance ramps at these new MSE walls, we estimate ultimate settlements on the order of 57 to 83 mm (2 to 3 inches). It is our opinion that these MSE Walls are capable of tolerating the anticipated settlement.

We understand deep foundations (precast concrete piles or drilled piers) will be provided for the wingwalls at the Gold Street bridge. Therefore settlement of these foundations is expected to be less than 8 mm (1/4 inch).

10.4 RESULTS OF SEEPAGE ANALYSES

In Section 6 "Seepage Evaluation" we discussed (1) method of analysis and (2) cross sections and results of analysis. In general, the levees are constructed of well compacted lean clays with some localized fat clays. Furthermore, most of the native soils underlying the levee fills are cohesive. Since the cohesive soils are relatively impervious, it is our engineering opinion that generally (1) seepage through and underneath the levees during the design flood will be relatively small and (2) the potential for piping at the outboard toe will also be very low. To confirm these opinions, we performed a seepage analysis on the six critical cross sections.

For a 30.5-meter (100-foot) long section, the total flow quantity through the levee at each of the six sections is expected to be less than 1/10 gallon per minute (gpm). In our opinion these seepage quantities are very small.

The above discussion is for a steady state condition such that seepage through the levee achieves equilibrium during the 100-year flood elevations. To simulate the change in river level during the flood event, we also performed a transient analysis based on the 100-year flood hydrograph. Based on this analysis, the highest phreatic surface is at either 36 or 48 hours; this surface is lower than the steady state phreatic surface. This observation is similar to the other five sections. By comparison, we conclude that the estimated seepage quantities probably will be much less than the 1/10 gpm estimated.

10.5 SITE PREPARATION AND GRADING

In Section 9 "Site Preparation and Earthwork" we discussed in detail the following earthwork aspects: (1) clearing and demolition, (2) grubbing, (3) stripping, (4) exploration trench, (5) removal of sediment in channel, (6) excavation, (7) subgrade preparation, (8) fill materials, including onsite materials and import materials, (9) fill placement and compaction and (10) construction dewatering. Please refer to that section for details.

10.6 LIQUEFACTION POTENTIAL

In Section 8 "Liquefaction Potential", we presented our updated evaluation of liquefaction potential of the site and concluded that there are locations that have a relatively high potential of liquefaction during major earthquake shaking. These locations are presented graphically in Figure 2. However, it is our opinion that lateral spreading due to liquefaction is unlikely. The likely consequence that may impact the levees will be liquefaction induced settlement; we estimate such settlement to be on the order of 12 to 110 mm (½ to 4½ inches). Therefore, we recommend that the District inspect the levee after major earthquake events to assess the need for raising the settled crest of the levee and/or grouting underneath floodwalls if gaps result from the levee settlement. In any event, the inspection and mitigation should be consistent with the District's Levee Safety Technical Guidance Manual.

The conclusions and recommendations contained in this report are based on exploratory borings drilled for this study, as well as our review of available subsurface information, our previous site reconnaissance, and our local experience and engineering judgment. Furthermore, the slope stability analysis has been limited to the six (6) critical cross sections selected.

Surveyed ground elevations of borings were not available. Elevations were estimated from nearby cross section geometry provided by the District and from spot elevations provided by CH2MHILL.

An investigation for subsurface environmental contamination was beyond the scope of our services.

If any variations or undesirable conditions are encountered during construction, URS should be contacted so that supplementary recommendations can be made. Also, if the construction plans are changed from those presently conceived, URS should review the changes and make modifications to the original recommendations presented in the report in order to meet the project needs.

Geotechnical issues may arise during construction which are not apparent at this time. URS should be retained during construction to review the soil conditions encountered and the construction procedures. All pile foundation installation and testing should be done under the direct observation of a URS representative.

The recommendations presented in this report were developed with the standard of care commonly used in this profession. No other warranties are included, either express or implied, as to the professional advice presented in this report.

SECTION 12

Abrahamson, N.A. and Silva, W.J., 1997, Empirical response spectral attenuation relations for shallow crustal earthquakes, Seismological Research Letters, v. 68, p. 94-127.

Boore, D.M., Joyner, W.B., and Fumal, T.E., 1993, Estimation of response spectra and peak accelerations from western North American earthquakes: An interim report, U.S. Geological Survey Open-File Report 93-509, 72 p.

Bureau of Reclamation, U.S. Department of the Interior, 1974, Earth Manual, Second Edition.

Campbell, K.W., 1997, Empirical near-source attenuation relationships for horizontal and vertical components of peak ground acceleration, peak ground velocity, and pseudo-absolute acceleration response spectra, Seismological Research Letters, v. 68, p. 154-179.

Cedergren H.R., 1967, Seepage, Drainage and Flow Notes, 489 p.

Department of the Army Corps of Engineers, 1970, Engineering and Design - Stability of Earth and Rock-Fill Dams, Manual EM-1110-2-1902.

Department of the Army Corps of Engineers, 1978, Design and Construction of Levees, Manual EM-1110-2-1913.

Department of the Army Corps of Engineers, April 30, 2000, Design and Construction of Levees, Manual EM-1110-2-1913.

Department of the Army Corps of Engineers, September 30, 1986, Seepage Analysis and Control of Dams, Manual EM-1110-2-1901.

Department of the Army Corps of Engineers, September 29, 1989, Retaining and Flood Walls.

Department of the Navy Naval Facilities Engineering Command, 1982, Soil Mechanics Design Manual 7.1.

Duncan, J.M. and Buchignani, A.L., 1975, An engineering manual for slope stability studies, University of California, Berkeley, Department of Civil Engineering.

Geo-Slope International, Ltd., 2001 User's Guide, SLOPE/W for Slope Stability Analysis, Version 4.24.

Geo-Slope International, Ltd., 1998, User's Guide, SEEP/W for Seepage Analysis, Version 4.21.

Hitchcock, C.S., Kelson, K.I. and Thompson, S.C., 1994, Geomorphic investigations of deformation along the northeastern margin of the Santa Cruz Mountains, William Lettis & Associates, Inc., Final Technical Report, U.S. Geological Survey National Earthquake Hazards Reduction Program Award No. 1434-92-G-2220, 33 p.

Irwin, W.P., 1990, Geology and plate tectonic development, in The San Andreas Fault System, California, Wallace, R.E. (ed.) U.S. Geological Survey Professional Paper 1515, p. 61-80.

Jennings, C.W., 1994, Fault activity map of California and adjacent areas, California Division of Mines and Geology, California Geologic Data Map Series, Map No. 6, 1:750,000 scale.

Leonards, G.A., 1962, Foundation Engineering, McGraw-Hill Book Company.

Lienkaemper, J.J., Borchardt, G. and Lisowski, M., 1991, Historic creep rate and potential for seismic slip along the Hayward fault, California, Journal of Geophysical Research, v. 96B, p. 18261-18283.

Lienkaemper, J.J. and Galehouse, J.S., 1997, Revised long-term creep rates on the Hayward fault, Alameda and Contra Costa Counties, California, U.S. Geological Survey, Open-File Report 97-690, 18 p.

Poland, J.F., 1971, Land Subsidence in the Santa Clara Valley, Alameda, San Mateo, and Santa Clara Counties, California, U. S. Geological Survey, Map MF - 336.

Rogers, T .H and Williams, J.W., 1974, Potential Seismic Hazards in Santa Clara County, California, California Division of Mines and Geology, Special Report 107, 39p.

Sadigh, K., Chang, C.Y., Egan, J.A., Makdisi, F. and Youngs, R.R., 1997, Attenuation relationships for shallow crustal earthquakes based on California strong motion data, Seismological Research Letters, v. 68, p. 180-189.

Seed, R.B. and Harder, L.F., May, 1990, SPT-Based analysis of cyclic pore pressure generation and undrained residual strength, Volume 2, H. Bolton Seed Memorial Symposium Proceedings.

URS Greiner Woodward Clyde, 2000. "Geotechnical Investigation, Lower Guadalupe River Flood Control Project, Santa Clara County, California." Job No. 95-973017NB.00. Dated April 28, 2000.

Wallace, R.E.(ed.), 1990, The San Andreas Fault System, California, USGS, Professional Paper 1515, 283 p.

Working Group on California Earthquake Probabilities, 1990, Probabilities of large earthquakes in the San Francisco Bay region, California, U.S. Geological Survey, Circular 1053, 51 p.

Working Group on Northern California Earthquake Potential, 1996, Database of potential sources for earthquakes larger than magnitude 6 in northern California, U.S. Geological Survey, Open-File Report 96-705, 53 p.

TABLES

TABLE 1
Lower Guadalupe River Flood Control Project
EXISTING BORING LOG INDEX LIST

Boring Index	Consultants	Consultants' Project Number	Project Date	Project Name
AA-n	Woodward-Clyde	973061NA	Sept 1997	Preliminary Investigation, Fuel Storage and Dispensing Facility for American Airlines, San Jose
BSP-n	Woodward-Clyde	4327	Dec 1960	Soil Investigation for the Proposed Bay Shell Plant Storage Tanks, San Jose
GJJ-n	Gribaldo,Jacobs, Jones & Associates	E584-M1	Jan 1963	Soil Investigation for Guadalupe River Project, Reach No. 1, Preliminary Study of the West Bank of Alviso Slough and Proposed SPRR Retaining Wall, Santa Clara County
GRI-n	John V. Lowney & Associates	109-11 PA 9669	Jan 1981	Geotechnical Investigation, Guadalupe River Improvement, San Jose
GRIM-n	Woodward-Clyde	15482V	Oct 1982	Geotechnical Investigation, Guadalupe River Flood Control Improvements, Southern Pacific Railroad to the County Marina in Alviso
GRIV-n	Woodward-Clyde	4246	c. 1960	Soil Investigation for the Proposed Guadalupe River Project, Reaches 1 and 2, Santa Clara County
GROCS-n	John V. Lowney & Associates	109-12 PA 11516	June 1983	Geotechnical Investigation, Guadalupe River Outfall Control Structure, San Jose
GSB-n	Woodward-Clyde	16338V	Oct 1984	Geotechnical Investigation, Raising of Gold Street Bridge over the Guadalupe River in Alviso, San Jose
GSBOLD-n	Woodward-Clyde	4246	c. 1960	Gold Street Bridge Investigation as part of the 1960 Guadalupe River Project
GTPP-n	Parikh Consultants, Inc.	953009NE	c. 1996	Golden Triangle Pipeline Project, South Bay Water Recycling Program
HHP-n	Woodward-Clyde	91C0639R	Jan 1992	"Hetch-Hetchy Pipelines," Relocation of San Francisco Water Department, Bay Division Pipeline Numbers 3 and 4 at Guadalupe River and Coyote Creek, San Jose and Santa Clara
HHPA-n	PSC Associates, Inc.	88122.10	July 1988	Lick Mill Blvd. Extension, Santa Clara
OPS-n	Woodward-Clyde	16784V	c. 1985	Geotechnical Consultation, Oakmead Storm Pump Station Outfall Spillway

TABLE 1
Lower Guadalupe River Flood Control Project
EXISTING BORING LOG INDEX LIST

Boring Index	Consultants	Consultants' Project Number	Project Date	Project Name
PRMAR-n	Soil Mechanics & Foundation Engineers	1140	Feb 1962	Soil Engineering Report for the Proposed Marina, Alviso, Santa Clara County
PYH-n	Cooper & Clark	33A	Jan 1960	Preliminary Site Investigation, Proposed Yacht Harbor, Alviso
RBPS-n	Woodward-Clyde	16171X	Feb 1987	Geotechnical Investigation, Retention Basin and Pump Station, San Jose International Airport, San Jose
REAL1-n	Woodward-Clyde	5135	June 1962	Soil Investigation, Guadalupe River Project, Realignment within Reach 1, Santa Clara
RHM-n	Woodward-Clyde	92C0526R	Nov 1992	Soil Investigation, Riparian Habitat Mitigation, Route 87
RREMB-n	John V. Lowney & Associates	109-1 PA 1233	Oct 1969	Soil Mechanics Investigation, SPRR Bridge and Embankment Raising over Guadalupe River, San Jose
SBYC-n	Woodward-Clyde	15482-W-2000	April 1983	Geotechnical Study, South Bay Yacht Club Relocation, Alviso
SDB-n	Woodward-Clyde	963010NA	c. 1997	Geotechnical Investigation, Skyport Drive Bridge, San Jose
SIL-n	Cooper & Clark	170-G	Oct 1982	Soil Investigation, Landslide Areas, Guadalupe River, San Jose
SJIA-n	Woodward-Clyde	16171V	Feb 1984	Geotechnical Investigation, San Jose International Airport, San Jose
SPRR-n	Jo Crosby & Associates	2706	Jan 1976	Soils Investigation for Guadalupe River, SPRR to Trimble Road, Project # 3015
SPRRB-n	Woodward-Clyde	4246	Jan 1961	Soil Investigation for the Proposed Southern Pacific Company Bridge, Alviso (in conjunction with soil investigation of Guadalupe River Project, Reaches 1 and 2)
TDB-n	Woodward-Clyde	16266V	May 1984	Geotechnical Study, Tasman Drive Bridge at Guadalupe River, Santa Clara/San Jose

Note:

1. The value "n" in the boring index column represents the Boring Number used in the corresponding project, such as 1 and B1.

TABLE 2a

ESTIMATED 84th PERCENTILE PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 1

Fault	Maximum Magnitude (M_w)	Sense of Faulting[1]	Distances			Peak Ground Acceleration (g)				
			Horizontal Distance[2] (km)	Seismogenic Distance[3] (km)	Rupture Distance[4] (km)	Abrahamson & Silva (1997)	Campbell (1997)	Sadigh *et al.* (1997)	Boore *et al.* (1997)	**Average**
Monte Vista - Shannon	6.8	R	13.8	15.9	13.8	0.45	0.46	0.48	0.45	**0.46**
Hayward - SE Extension	6.5	OR	9.4	11.6	9.4	0.57	0.56	0.58	0.50	**0.55**
Hayward	7.1	SS	10.6	10.8	10.6	0.46	0.58	0.48	0.56	**0.52**
Calaveras	6.7	SS	13.8	14.0	13.8	0.36	0.42	0.37	0.38	**0.38**
San Andreas - Peninsula Segment	7.2	SS	20	20.1	20	0.30	0.40	0.33	0.38	**0.36**
San Andreas - Santa Cruz Mountains Segment	7.2	SS	20	20.1	20	0.30	0.40	0.33	0.38	**0.36**
San Andreas - 1906 Rupture	8	SS	20	20.1	20	0.38	0.58	0.45	0.58	**0.50**
Sargent	6.8	OR	17.8	19.9	17.8	0.37	0.36	0.40	0.38	**0.38**

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] M_w and horizontal distance used by Boore *et al.* (1997). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] M_w and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] M_w and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

TABLE 2b

ESTIMATED 84th PERCENTILE PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 2

Fault	Maximum Magnitude (M_w)	Sense of Faulting[1]	Distances			Peak Ground Acceleration (g)				
			Horizontal Distance[2] (km)	Seismogenic Distance[3] (km)	Rupture Distance[4] (km)	Abrahamson & Silva (1997)	Campbell (1997)	Sadigh *et al.* (1997)	Boore *et al.* (1997)	**Average**
Monte Vista - Shannon	6.8	R	14.4	16.4	14.4	0.43	0.45	0.47	0.43	**0.45**
Hayward - SE Extension	6.5	OR	9.4	11.6	9.4	0.57	0.56	0.58	0.50	**0.55**
Hayward	7.1	SS	11.9	12.1	11.9	0.42	0.55	0.45	0.52	**0.48**
Calaveras	6.7	SS	13.1	13.3	13.1	0.38	0.44	0.38	0.39	**0.40**
San Andreas - Peninsula Segment	7.2	SS	20	20.1	20	0.30	0.40	0.33	0.38	**0.36**
San Andreas - Santa Cruz Mountains Segment	7.2	SS	20	20.1	20	0.30	0.40	0.33	0.38	**0.36**
San Andreas - 1906 Rupture	8	SS	20	20.1	20	0.38	0.58	0.45	0.58	**0.50**
Sargent	6.8	OR	17.5	19.6	17.5	0.37	0.37	0.41	0.38	**0.38**

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] M_w and horizontal distance used by Boore *et al.* (1997). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] M_w and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] M_w and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

TABLE 2c

ESTIMATED 84th PERCENTILE PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 3

Fault	Maximum Magnitude (M_w)	Sense of Faulting[1]	Distances			Peak Ground Acceleration (g)				
			Horizontal Distance[2] (km)	Seismogenic Distance[3] (km)	Rupture Distance[4] (km)	Abrahamson & Silva (1997)	Campbell (1997)	Sadigh *et al.* (1997)	Boore *et al.* (1997)	**Average**
Monte Vista - Shannon	6.8	R	13.8	15.9	13.8	0.45	0.46	0.48	0.45	**0.46**
Hayward - SE Extension	6.5	OR	9.4	11.6	9.4	0.57	0.56	0.58	0.50	**0.55**
Hayward	7.1	SS	13.8	13.9	13.8	0.38	0.50	0.41	0.47	**0.44**
Calaveras	6.7	SS	13.8	13.9	13.8	0.36	0.42	0.37	0.38	**0.38**
San Andreas - Peninsula Segment	7.2	SS	19.5	19.6	19.5	0.31	0.41	0.34	0.39	**0.36**
San Andreas - Santa Cruz Mountains Segment	7.2	SS	19.5	19.6	19.5	0.31	0.41	0.34	0.39	**0.36**
San Andreas - 1906 Rupture	8	SS	19.5	19.6	19.5	0.39	0.59	0.46	0.59	**0.51**
Sargent	6.8	OR	16.9	19.0	16.9	0.38	0.38	0.42	0.39	**0.39**

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] M_w and horizontal distance used by Boore *et al.* (1997). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] M_w and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] M_w and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

TABLE 2d

ESTIMATED 84th PERCENTILE PEAK HORIZONTAL ACCELERATIONS IN g's AT SEGMENT 4

Fault	Maximum Magnitude (M_w)	Sense of Faulting[1]	Distances			Peak Ground Acceleration (g)				
			Horizontal Distance[2] (km)	Seismogenic Distance[3] (km)	Rupture Distance[4] (km)	Abrahamson & Silva (1997)	Campbell (1997)	Sadigh *et al.* (1997)	Boore *et al.* (1997)	Average
Monte Vista - Shannon	6.8	R	12.5	14.6	12.5	0.48	0.50	0.52	0.48	**0.49**
Hayward - SE Extension	6.5	OR	9.4	11.6	9.4	0.57	0.56	0.58	0.50	**0.55**
Hayward	7.1	SS	16.3	16.4	16.3	0.34	0.45	0.37	0.42	**0.40**
Calaveras	6.7	SS	13.1	13.3	13.1	0.38	0.44	0.38	0.39	**0.40**
San Andreas - Peninsula Segment	7.2	SS	18.8	18.9	18.8	0.32	0.42	0.35	0.40	**0.37**
San Andreas - Santa Cruz Mountains Segment	7.2	SS	18.8	18.9	18.8	0.31	0.41	0.34	0.39	**0.36**
San Andreas - 1906 Rupture	8	SS	18.8	18.9	18.8	0.31	0.41	0.34	0.39	**0.36**
Sargent	6.8	OR	16.3	18.4	16.3	0.39	0.40	0.43	0.40	**0.40**

[1] R - reverse or thrust fault; OR - oblique-reverse fault; SS - strike-slip fault.

[2] M_w and horizontal distance used by Boore *et al.* (1997). Horizontal distance is defined as the shortest distance from the site to the vertical projection of the fault rupture on the earth's surface.

[3] M_w and seismogenic distance used by Campbell (1997). Seismogenic distance is the shortest distance from the site to the zone of seismogenic rupture. The top of this zone is assumed to be at a depth of 2 km.

[4] M_w and rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997). Rupture distance is the shortest distance from the site to the surface rupture.

TABLE 3
Lower Guadalupe River Flood Control Project
SUBSURFACE CHARACTERIZATION
LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT

Levee	Levee Section Number	Station Limits		Comments
		From	To	
West	W1	7+000	7+450	
	W2	7+450	8+200	Common with E2
	W3	8+200	8+500	
	W4	8+500	9+230	
	W5	9+230	10+100	
	W6	10+100	11+730	
	W7	11+730	13+000	
	W8	13+000	14+040	
	W9	14+040	14+570	Common with E9
	W10	14+570	15+450	Common with E10
	W11	15+450	15+900	
	W12	15+900	16+770	
	W13	16+770	17+550	
East	E1	6+200	6+640	
	E2	6+640	8+050	Common with W2
	E3	8+050	9+550	
	E4	9+550	10+140	
	E5	10+140	11+100	
	E6	11+100	11+440	
	E7	11+440	12+450	
	E8	12+450	13+650	
	E9	13+650	14+850	Common with W9
	E10	14+850	15+56	Common with W10

Table 4
Lower Guadalupe River Flood Control Project
PROPOSED 6 CROSS SECTIONS OF LEVEES CONSIDERED TO BE MOST CRITICAL

Cross Section Number	Relevant Levee Section Number	Approximate Station	Section Location	Relevant Boring(s)	Comments
8	E2	7+170	East Levee	CB-01, EC-01, GROCS-1, CB-01	loose sand, levee raise
9	W1	7+170	West Levee	EB-19, GSB-5, GRI-EB4, EB-18, EB-19, EB-20	levee raise
10	W1	7+320	West Levee	EB-20, WB-01	loose sand, levee raise
11	E2	7+800	East Levee	EB-03, WB-19	loose sand, levee raise
12	W2	7+950	West Levee	WB-18, WB-19, WB-20	VTA Mitigation Levee
13	E2	7+997	East Levee	EB-04, EB-05, EB-06, EC-03, EC-04, EB-21, EB-22	soft clay in embankment, levee raise

TABLE 5
Lower Guadalupe River Flood Control Project
SUMMARY OF EFFECTIVE SHEAR STRENGTH PARAMETERS
BASED ON ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL TESTS

SOIL TYPE	DRY DENSITY RANGE (pcf)	ESTIMATED EFFECTIVE STRENGTH PARAMETERS							
		ϕ' (degrees)				c' (psf)			
		Peak	10% ε	20% ε	Design	Peak	10% ε	20% ε	Design
Lean CLAY FILL	Less than 110	23.7	22	16.3	24	456	466	574	450
Lean CLAY FILL	Greater than 110	32.2	33.8	30.1	33	0	0	0	0
Fat CLAY	Less than 80	19.8	12.2	NA	20	113	214	NA	110
Lean CLAY	From 80 to 100	30.4	29	26.7	30	35.1	182.6	186.5	35
Lean CLAY	From 100 to 110	29.4	30	25.3	30	153.4	144.8	424.2	150

Note: Definitions for attached figures are

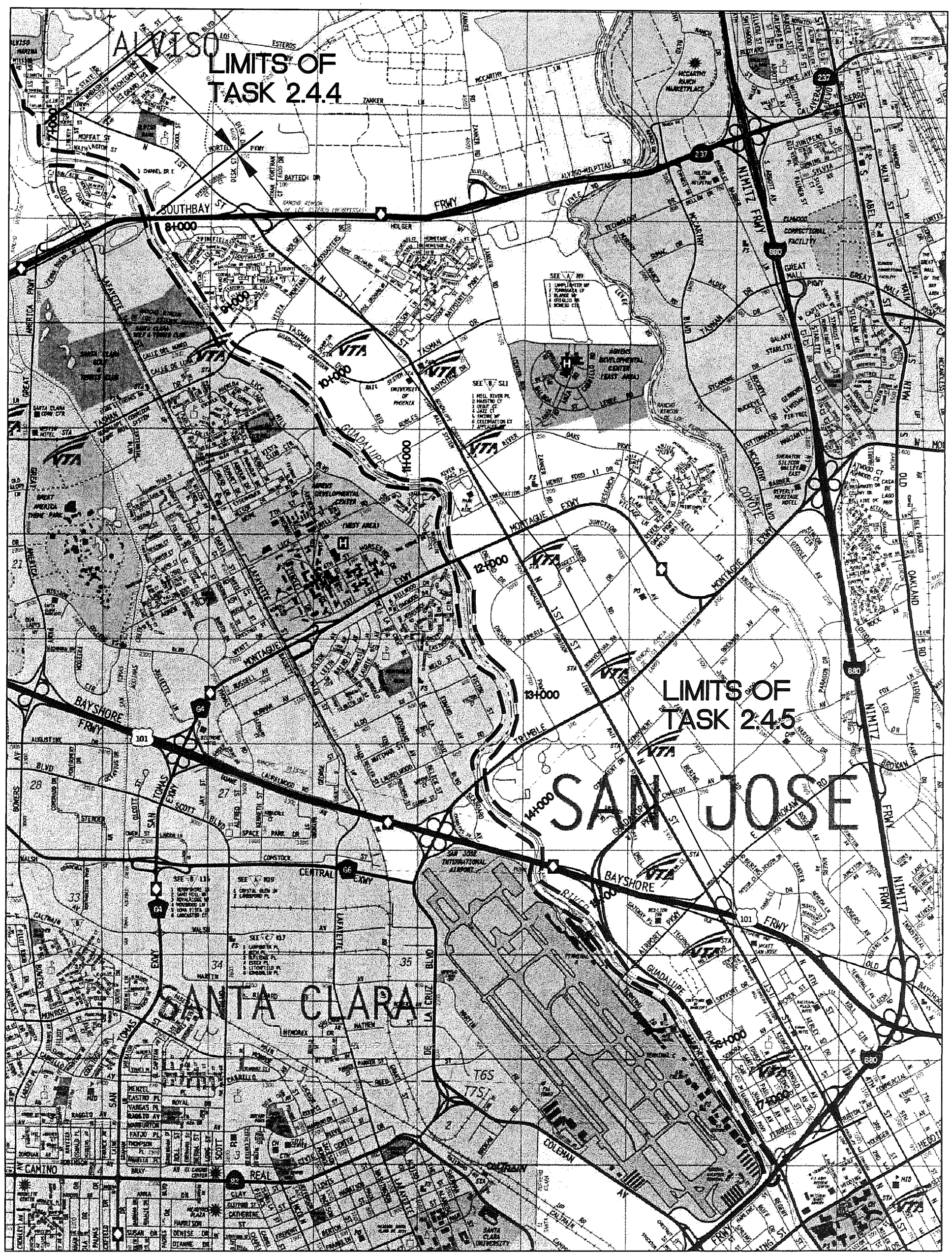
$q = ½ (\sigma_1' - \sigma_3')$

$p = ½ (\sigma_1' + \sigma_3')$

TABLE 6
Lower Guadalupe River Flood Control Project
ARMY CORPS OF ENGINEERS
SLOPE STABILITY DESIGN REQUIREMENTS

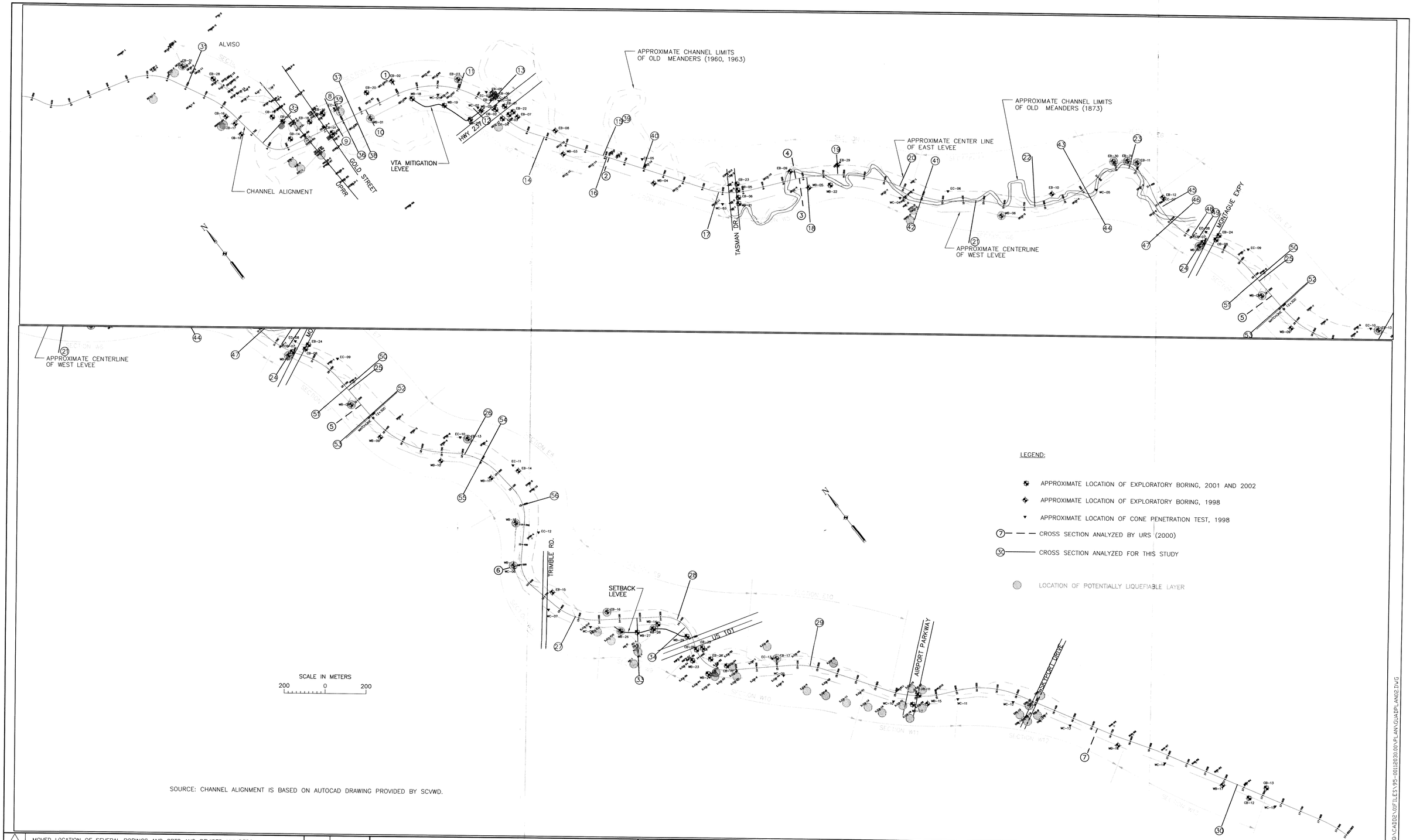
Case Number	Design Condition	Slope Analyzed	Shear Strength	Minimum Required Factor of Safety
I	End of Construction	Riverside and Landside	Q or S	1.3
II	Sudden Drawdown	Riverside	S where < R R where < S	1.0
III	Steady Seepage from Full Flood Stage	Riverside and Landside	S where < R R+S/2 where R<S	1.4
IV	Earthquake	Riverside and Landside	Q or S	Not applicable

FIGURES



NOT TO SCALE

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	SITE MAP	Figure 1
6/1/02			
URS			



SOURCE: CHANNEL ALIGNMENT IS BASED ON AUTOCAD DRAWING PROVIDED BY SCVWD.

REV	DESCRIPTION OF REVISION	BY	DATE
5	MOVED LOCATION OF SEVERAL BORINGS AND CPTS AND REVISED NUMBERS.	S.H.	3/19/98
4	MOVED WC-4 AND WB-5 PER CULTURAL RESOURCE RECOMMENDATION.	S.H.	4/9/98
3	ADDED CROSS SECTIONS 8 THROUGH 30 AND LEVEE SECTIONS.	M.L.L.	9/18/01
2	ADDED BORING LOCATIONS (2001 STUDY) AND VTA MITIGATION LEVEE	J.W.	11/05/01
1	ADDED NEW BORING LOCATIONS, SETBACK LEVEE, CROSS SECTIONS 31 THROUGH 56	T.P.	5/29/02
0			

7/1/02

URS
55 South Market Street, Suite 1650
San Jose, Ca. 95113

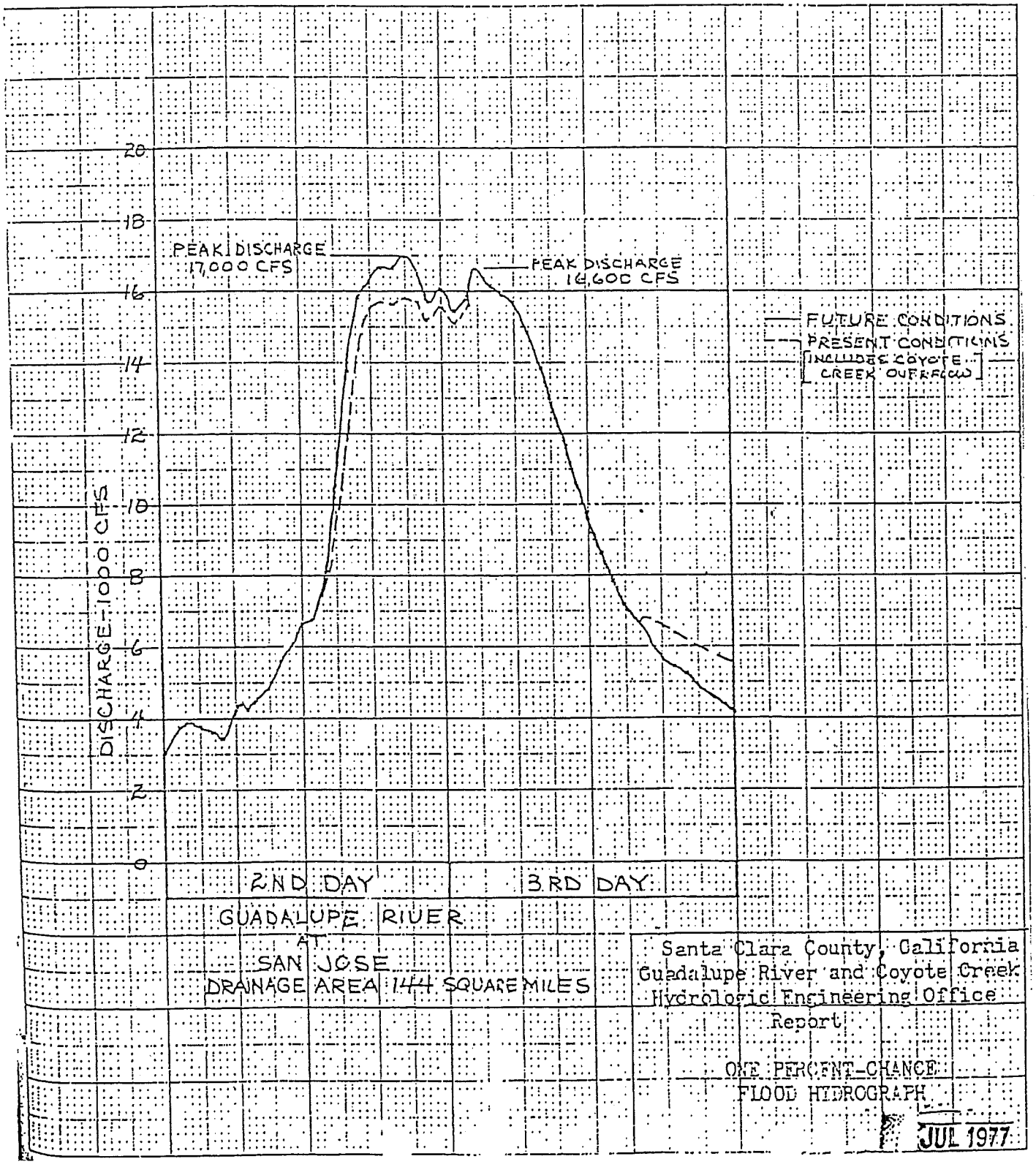
DESIGNED T.P.
DRAWN E.G.
CHECKED S.H.
PEER REVIEWED M.L.
PROJECT MANAGER P.B.
DATE 7/1/02

Lower Guadalupe River Flood Protection Project Santa Clara County, CA.

SITE AND FIELD EXPLORATION LOCATION PLAN

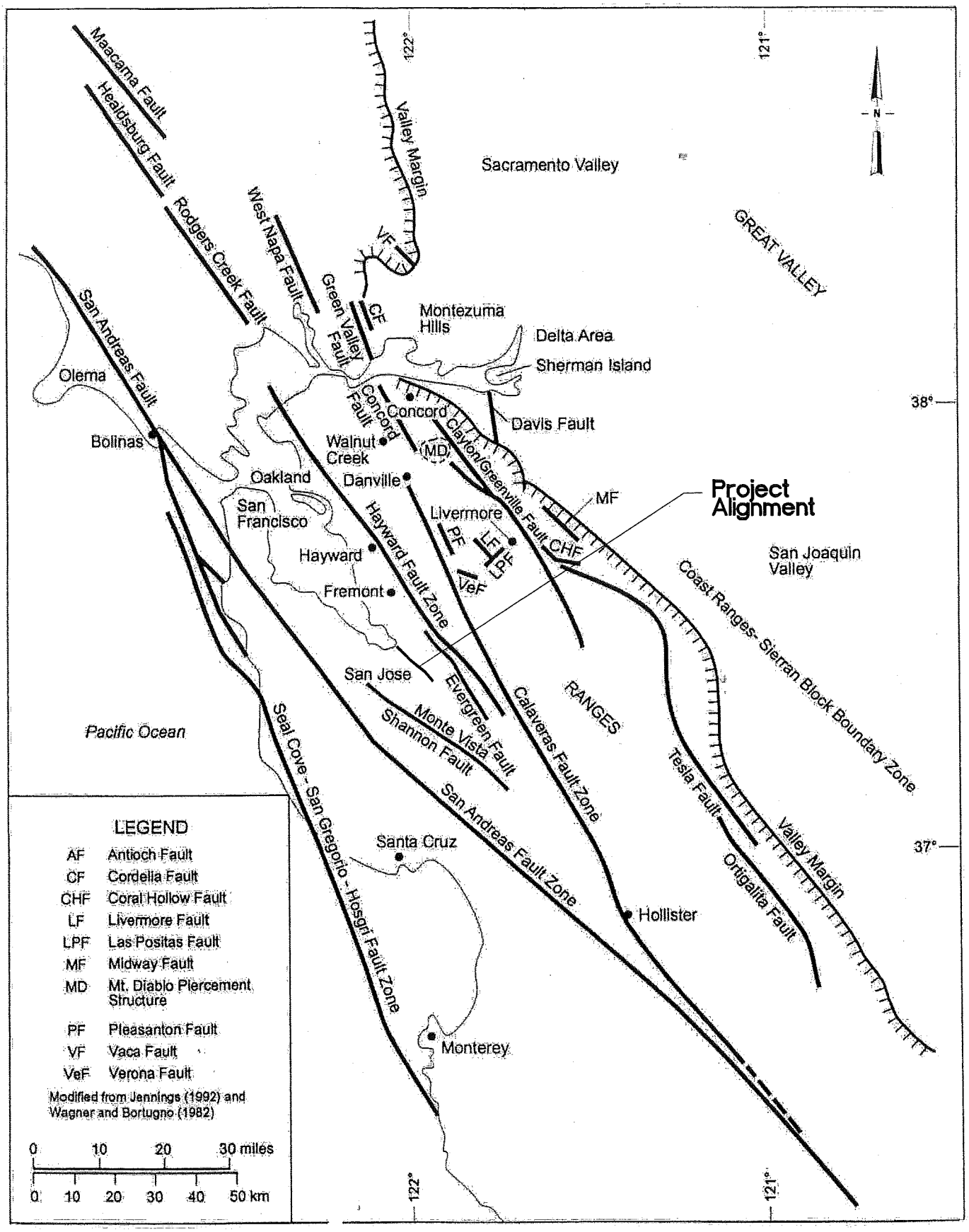
REVISION 5
PROJECT 95-00112030.00
DRAWING Fig. 2
SHEET 1 OF 1

G:\CADD2\JFILES\95-00112030.00\PLAN\GUADPLAN02.DWG



Q:\CADD2\01FILES\95-00112030.00\244\GUADFIG03.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	ONE PERCENT CHANCE FLOOD HYDROGRAPH	Figure 3
6/1/02	URS		



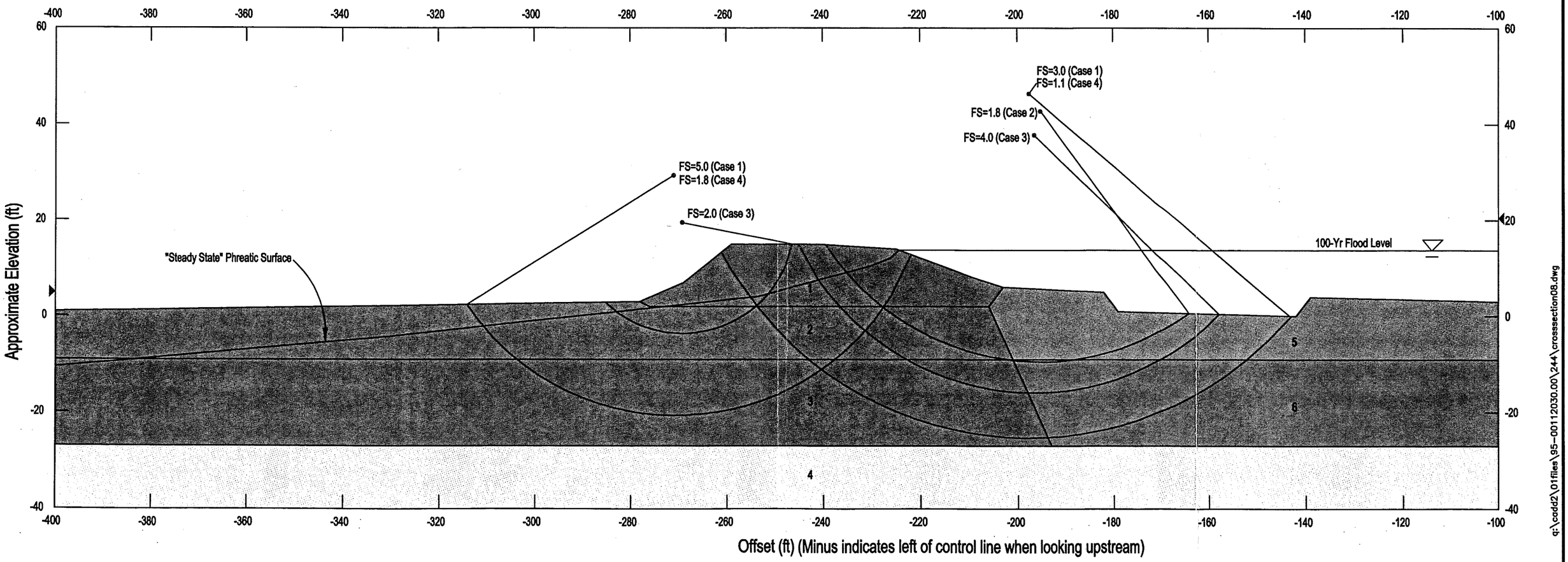
Q:\cadd2\01files\95-00112030.00\244\faultmap04.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	REGIONAL ACTIVE FAULTS	Figure 4
6/1/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 5 Figure 11	109 109	20 20	131 131	24 24	450 450	2000 1540	Selected
2	Fat CLAY	Table 5 Figure 11	85 85	60 60	136 136	20 20	110 110	1800 1800	Selected
3	Lean CLAY	Table 5 Figure 11	95 95	30 30	124 124	30 30	35 35	1200 1200	Selected
4	Silty SAND	Figure 11			135	33 33	0 0		Estimated from N=10, σ v' = 3.0 ksf, Dr = 50% Selected
5	Fat CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	80	47	118	20	110	400	Selected
6	Lean CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	90	30	118	30	35	425	Selected

Notes:

1. Elevations are based on NGVD datum.



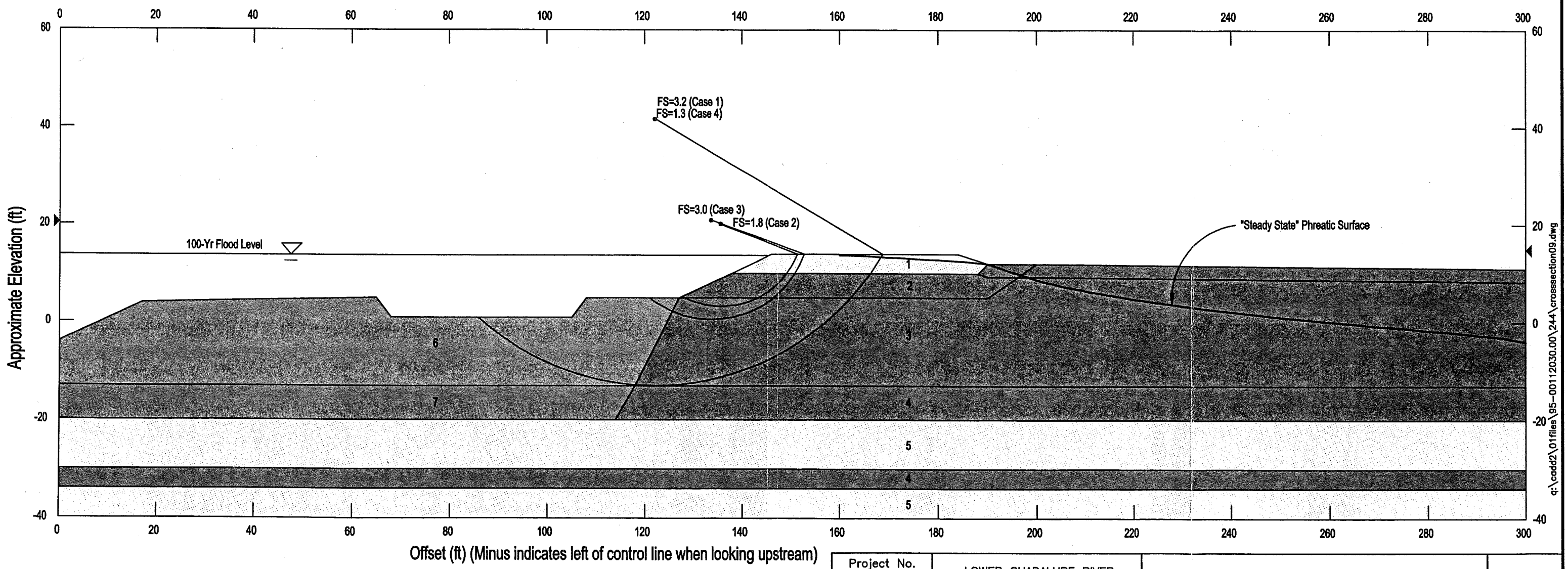
q:\cadd2\01files\95-00112030.00\244\crosssection08.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 8 STATION 7+170 EAST LEVEE	Figure 5
9/23/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Silty SAND (FILL)				135	33 33	0 0		Estimated from N = 10, σv' = 0.3 ksf, Dr = 70% Selected
2	Lean CLAY (FILL)	Table 5 Figure 12	109 109	18 18	129 129	24 24	450 450	3000 1880	Selected
3	Fat CLAY	Table 5 Figure 12	90 90	30 30	117 117	20 20	110 110	1000 1000	Selected
4	Lean CLAY	Table 5 Figure 12	100 100	30 30	130 130	30 30	150 150	2000 2000	Selected
5	Silty SAND				135	36 36	0 0		Estimated from N = 35, σv' = 3.3 ksf, Dr = 80% Selected
6	Fat CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	80	47	118	20	110	400	Selected
7	Lean CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	90	30	118	30	150	425	Selected

Notes:

1. Landside slope not analyzed due to relatively flat slope.
2. Elevations are based on NGVD datum.

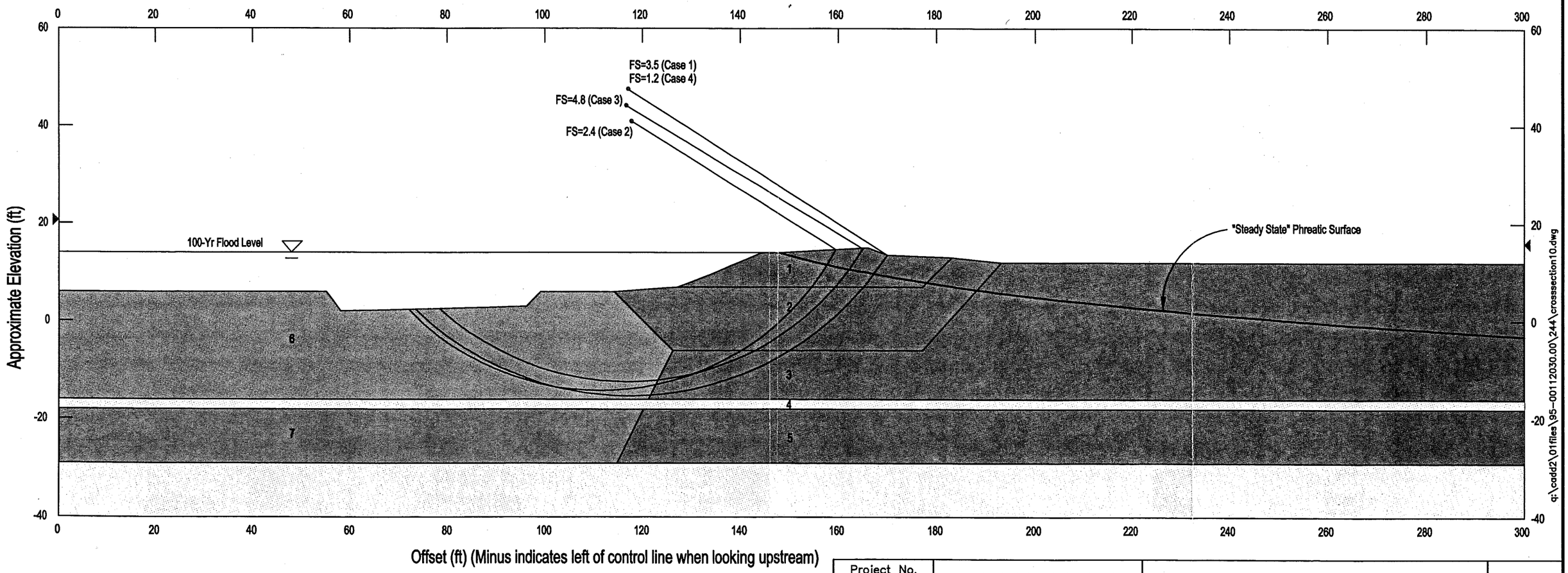


Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 9 STATION 7+170 WEST LEVEE	Figure 6
9/23/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 5				33	0		
		Figure 13	110	18	130			3000	
1	Lean CLAY (FILL)		110	18	130	33	0	1880	Selected
		Table 5				24	450		
		Figure 13	90	30	117			1000	
2	Fat CLAY (FILL)		90	30	117	24	450	1000	Selected
		Table 5				30	35		
		Figure 13	98	30	127			1000	
3	Lean CLAY		98	30	127	30	35	1000	Selected
						33	0		Estimated from N = 10,
									σ v' = 3.3 ksf, Dr = 50%
4	Silty SAND				135	33	0		Selected
		Table 5				30	35		
		Figure 13	90	30	117			1200	
5	Lean CLAY		90	30	117	30	35	1200	Selected
6	Fat CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	80	47	118	20	110	400	Selected
7	Lean CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	90	30	118	30	35	425	Selected

Notes:

1. Landside slope not analyzed due to relatively flat slope.
2. Elevations are based on NGVD datum.

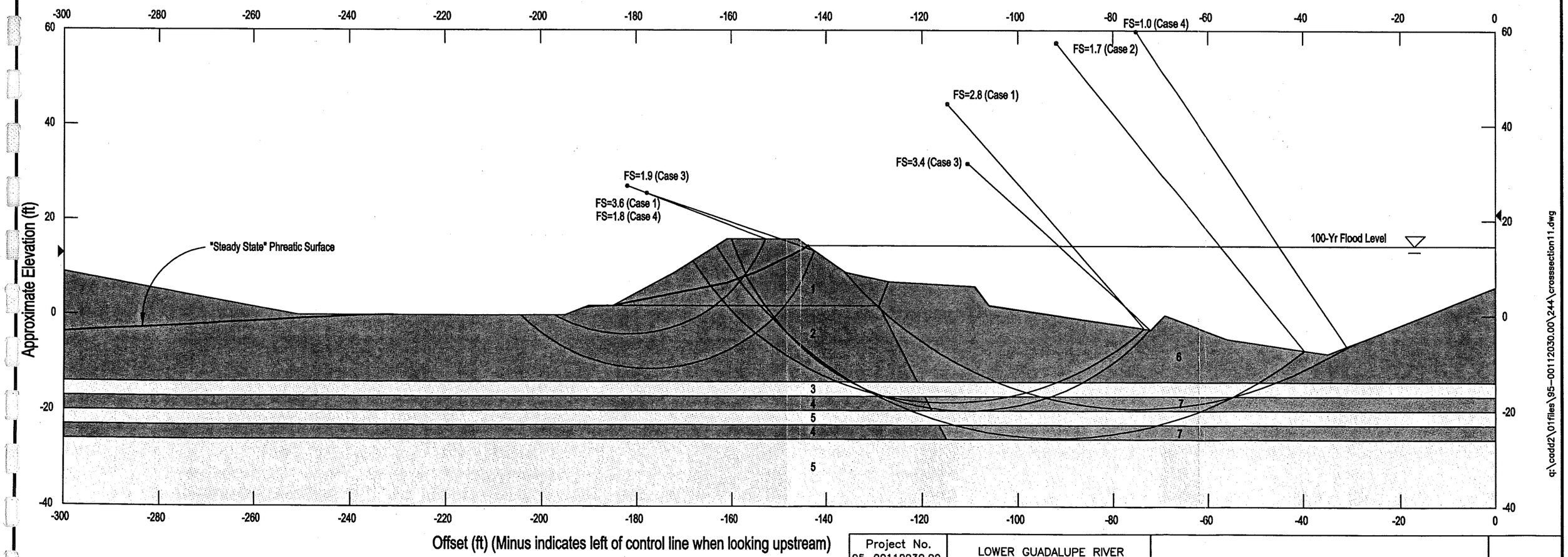


Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 10 STATION 7+320 WEST LEVEE	Figure 7
9/23/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 5 Figure 14	108 108	20 20	132 132	24 24	450 450	2000 1460	Selected
2	Lean CLAY	Table 5 Figure 14	100 100	30 30	130 130	30 30	150 150	1200 1200	Selected
3	Silty SAND				135	32 32	0 0		Estimated from N = 6, σv' = 1.6 ksf, Dr = 50% Selected
4	Fat Clay	Table 5 Figure 14	75 75	75 75	131 131	20 20	110 110	800 800	Selected
5	Silty SAND				135	34 34	0 0		Estimated from N = 12, σv' = 2.5 ksf, Dr = 60% Selected
6	Lean CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	90	30	118	30	150	425	Selected
7	Fat CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	80	47	118	20	110	400	Selected

Notes:

1. Elevations are based on NGVD datum.

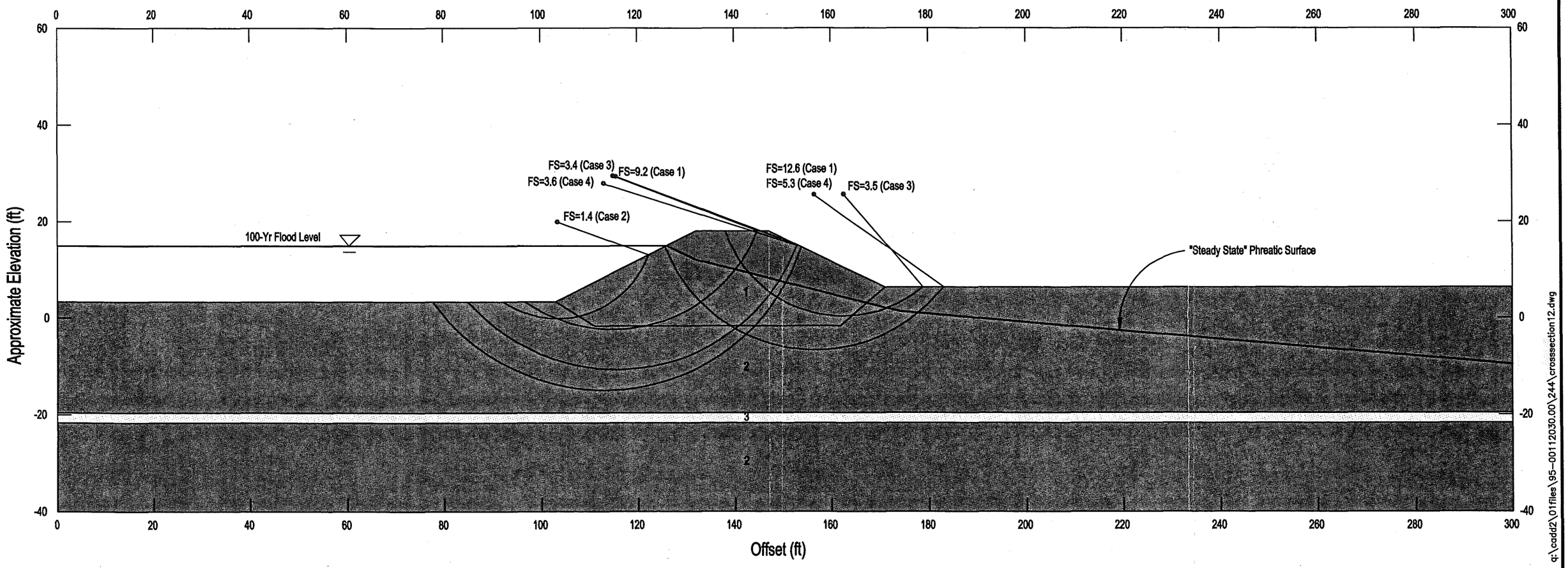


Project No. 95-00112030.00 9/23/02	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 11 STATION 7+800 EAST LEVEE	Figure 8
	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 5 WB-19, WB-20	99 99	19 19	118 118	24 24	450 450	4000 2800	Selected
2	Lean CLAY	Table 5 WB-19, WB-20	101 101	23 23	124 124	30 30	150 150	2000 2000	Selected
3	Silty SAND	WB-19, WB-20			135	34	0		Selected

Notes:

1. Elevations are based on NGVD datum.

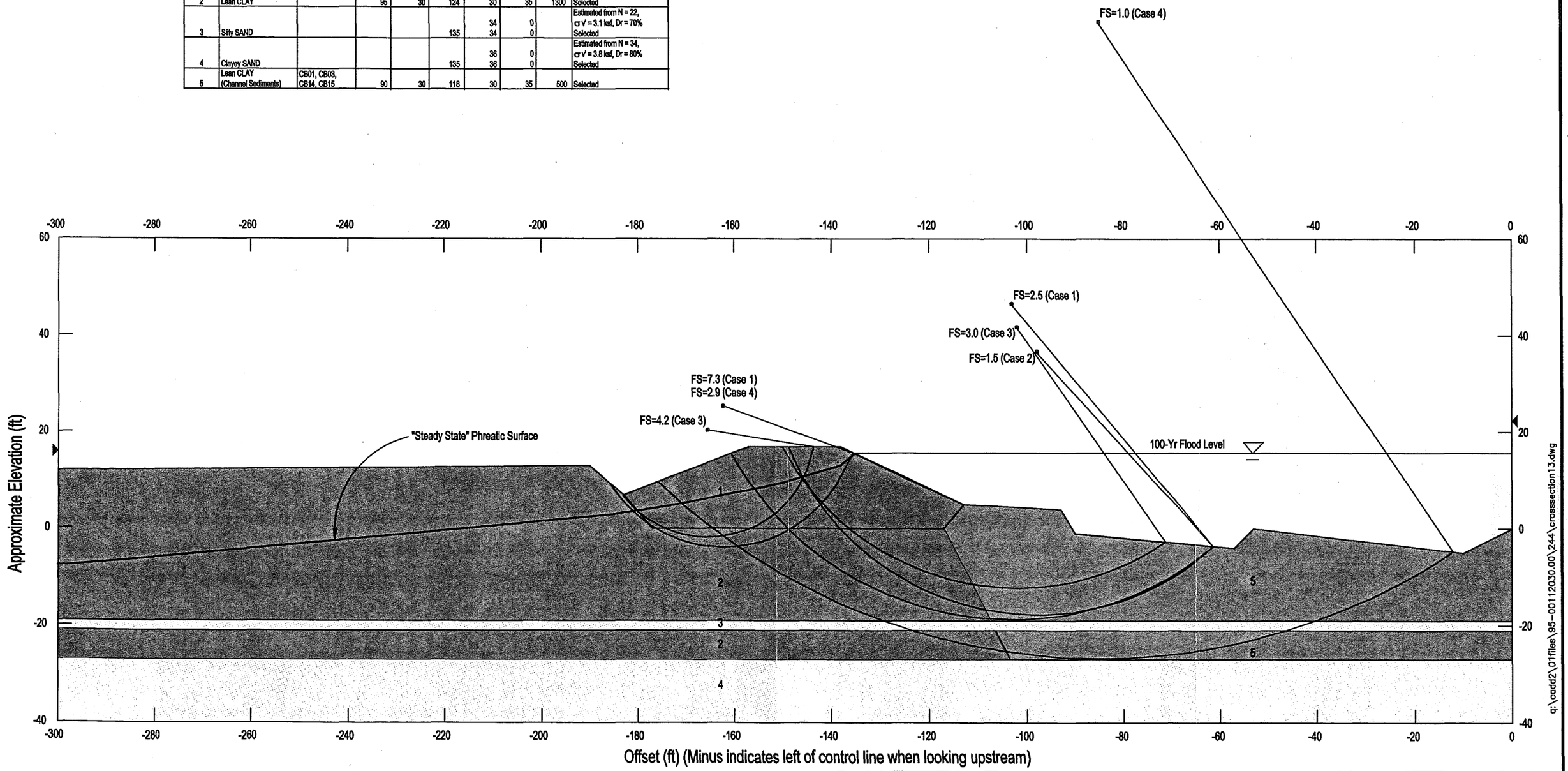


Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 12 STATION 7+950 WEST LEVEE	Figure 9
9/23/02	URS		

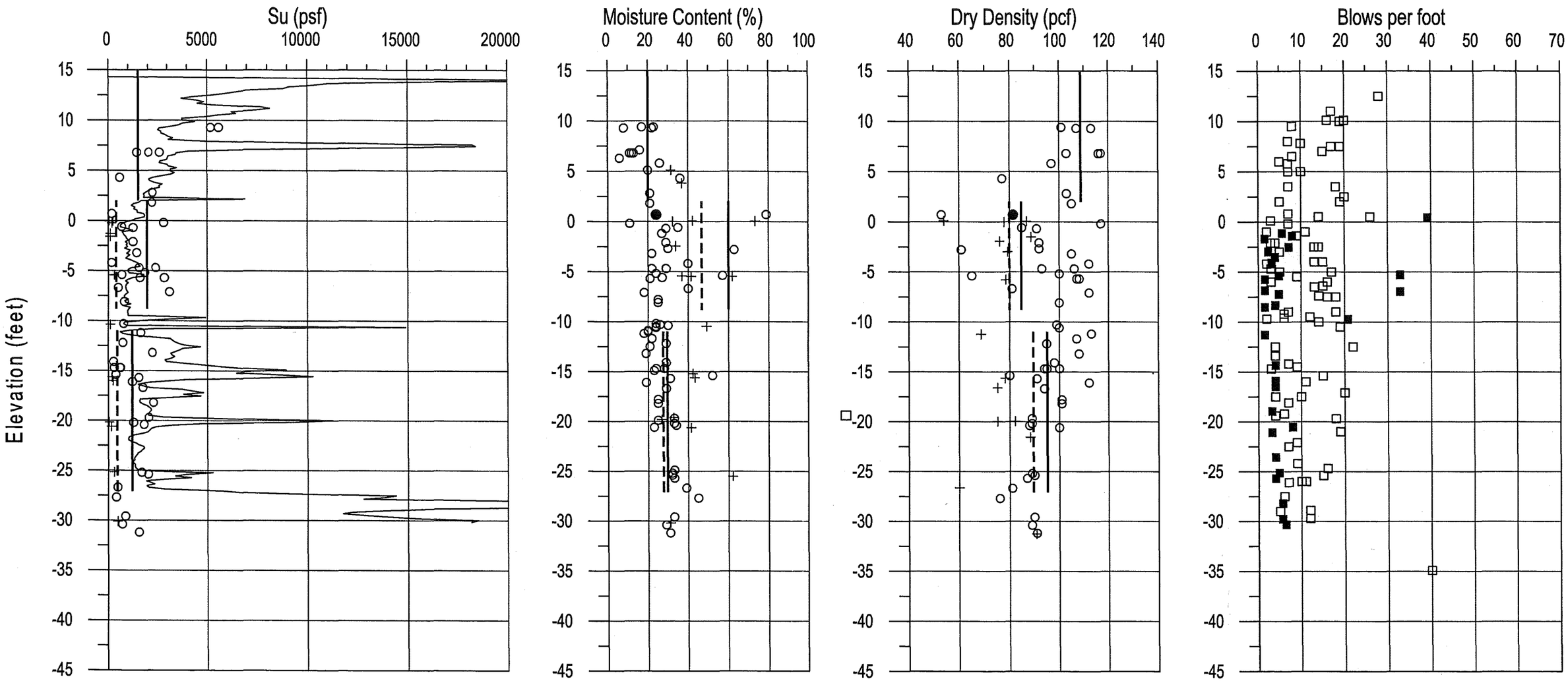
Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 5				24	450		
		Figure 16	108	20	132			2000	
1	Lean CLAY (FILL)		108	20	132	24	450	1420	Selected
		Table 5				30	35		
		Figure 16	95	30	124			1300	
2	Lean CLAY		95	30	124	30	35	1300	Selected
						34	0		Estimated from N = 22, σ v' = 3.1 ksf, Dr = 70%
3	Silty SAND				135	34	0		Selected
						36	0		Estimated from N = 34, σ v' = 3.8 ksf, Dr = 80%
4	Clayey SAND				135	36	0		Selected
5	Lean CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	90	30	118	30	35	500	Selected

Notes:

1. Elevations are based on NGVD datum.



Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS CROSS SECTION 13 STATION 7+997 EAST LEVEE	Figure 10
9/23/02	URS		



Data from the following borings and CPTs:

GRI-EB2	GSB-4	RREMB-5	EB-19
GRI-EB3	GSB-5	RREMB-6	EB-20
GRI-EB4	GSBOLD-1	SPRRB-R2	WB-01
GRI-EB5	GSBOLD-2	SPRRB-R3	WB-02
GRIV-21	GSBOLD-3	SPRRB-R4	CB-01
GROCS-1	GSBOLD-4	SPRRB-R5	CB-14
GSB-1	GSBOLD-5	SPRRB-R6	CB-15
GSB-2	GSBOLD-6	SPRRB-R7	EC-01
GSB-3	RREMB-4	EB-18	

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

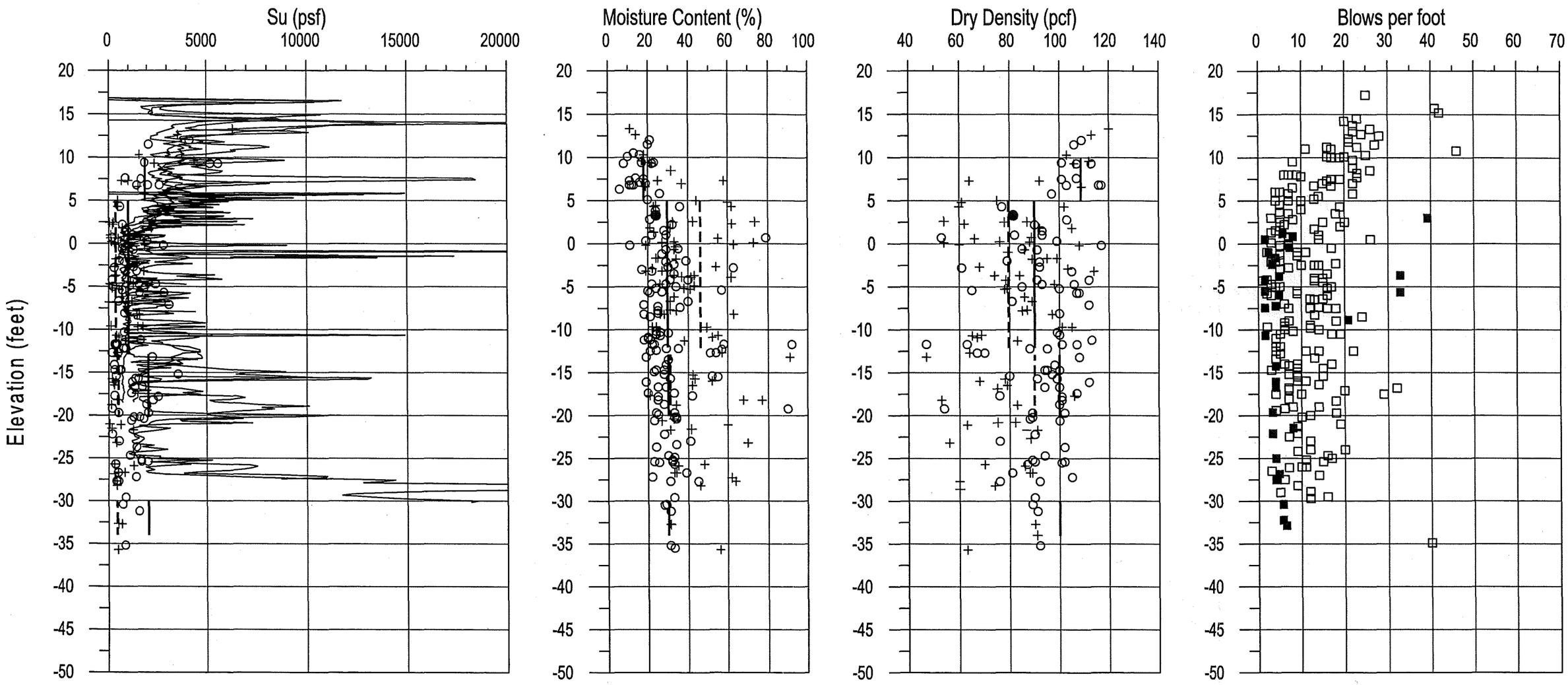
LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\244\index08.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 8	Figure 11
6/1/02	URS		



Data from the following borings and CPTs:

BSP-1	GRI-EB6	GRIV-21	GSBOLD-2	RREMB-6	EB-04	WB-18	WC-01
BSP-2	GRI-EB7	GROCS-1	GSBOLD-3	SPRRB-R1	EB-05	WB-19	WC-02
BSP-3	GRI-EB8	GSB-1	GSBOLD-4	SPRRB-R2	EB-06	WB-20	
GJJ-6	GRI-EB9	GSB-2	GSBOLD-5	SPRRB-R3	EB-07	CB-01	
GRI-EB1	GRIM-6	GSB-2A	GSBOLD-6	SPRRB-R4	EB-18	CB-14	
GRI-EB10	GRIM-7	GSB-2B	RREMB-1	SPRRB-R5	EB-19	CB-15	
GRI-EB2	GRIV-17	GSB-3	RREMB-2	SPRRB-R6	EB-20	EC-01	
GRI-EB3	GRIV-18	GSB-4	RREMB-3	SPRRB-R7	EB-27	EC-02	
GRI-EB4	GRIV-19	GSB-5	RREMB-4	EB-02	WB-01	EC-03	
GRI-EB5	GRIV-20	GSBOLD-1	RREMB-5	EB-03	WB-02	EC-04	

Levee Fill and Foundation Soil

+ Fat Clay
○ Lean Clay
□ All Soil Types

Channel Sediments

+ Fat Clay
● Lean Clay
■ All Soil Types

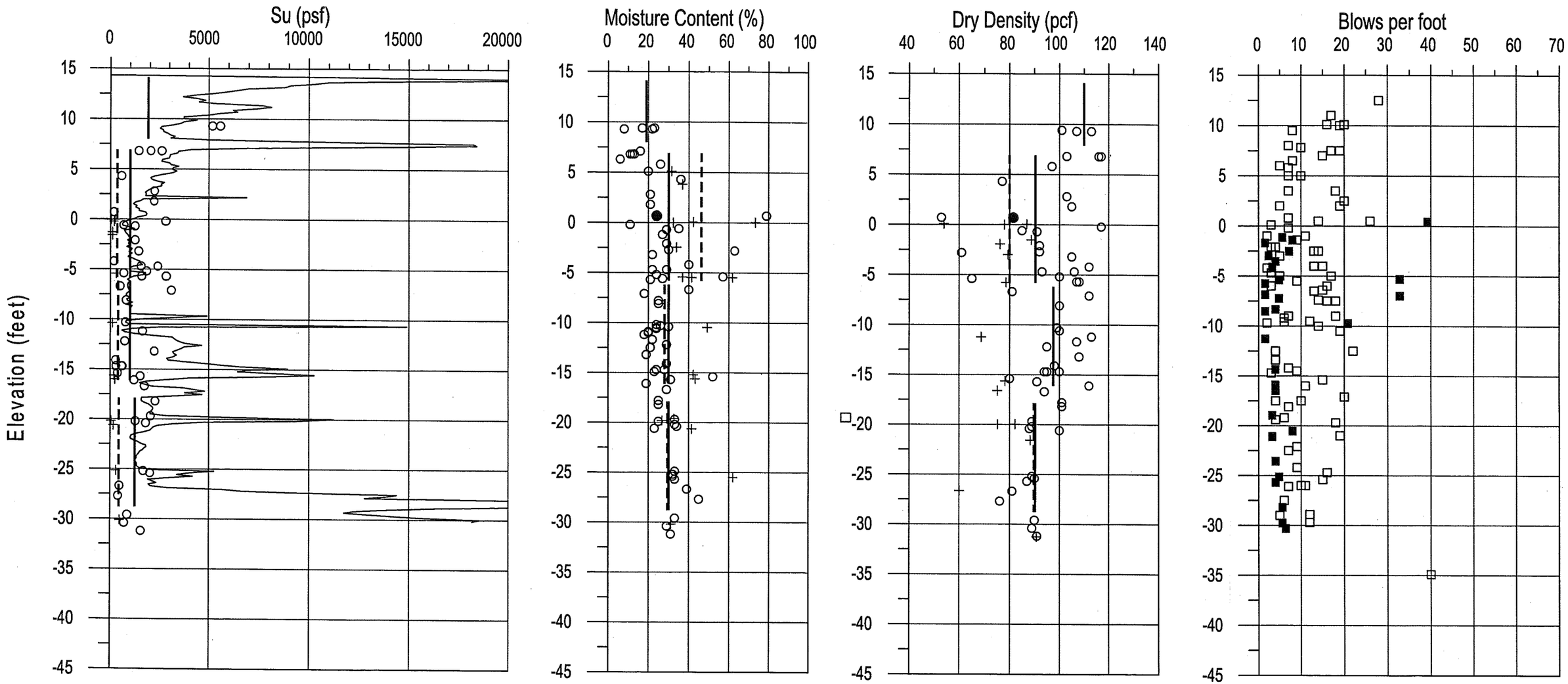
LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\244\index09.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 9	Figure 12
6/1/02	URS		



Data from the following borings and CPTs:

GRI-EB2	GSB-4	RREMB-5	EB-19
GRI-EB3	GSB-5	RREMB-6	EB-20
GRI-EB4	GSBOLD-1	SPRRB-R2	WB-01
GRI-EB5	GSBOLD-2	SPRRB-R3	WB-02
GRIV-21	GSBOLD-3	SPRRB-R4	CB-01
GROCS-1	GSBOLD-4	SPRRB-R5	CB-14
GSB-1	GSBOLD-5	SPRRB-R6	CB-15
GSB-2	GSBOLD-6	SPRRB-R7	EC-01
GSB-3	RREMB-4	EB-18	

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

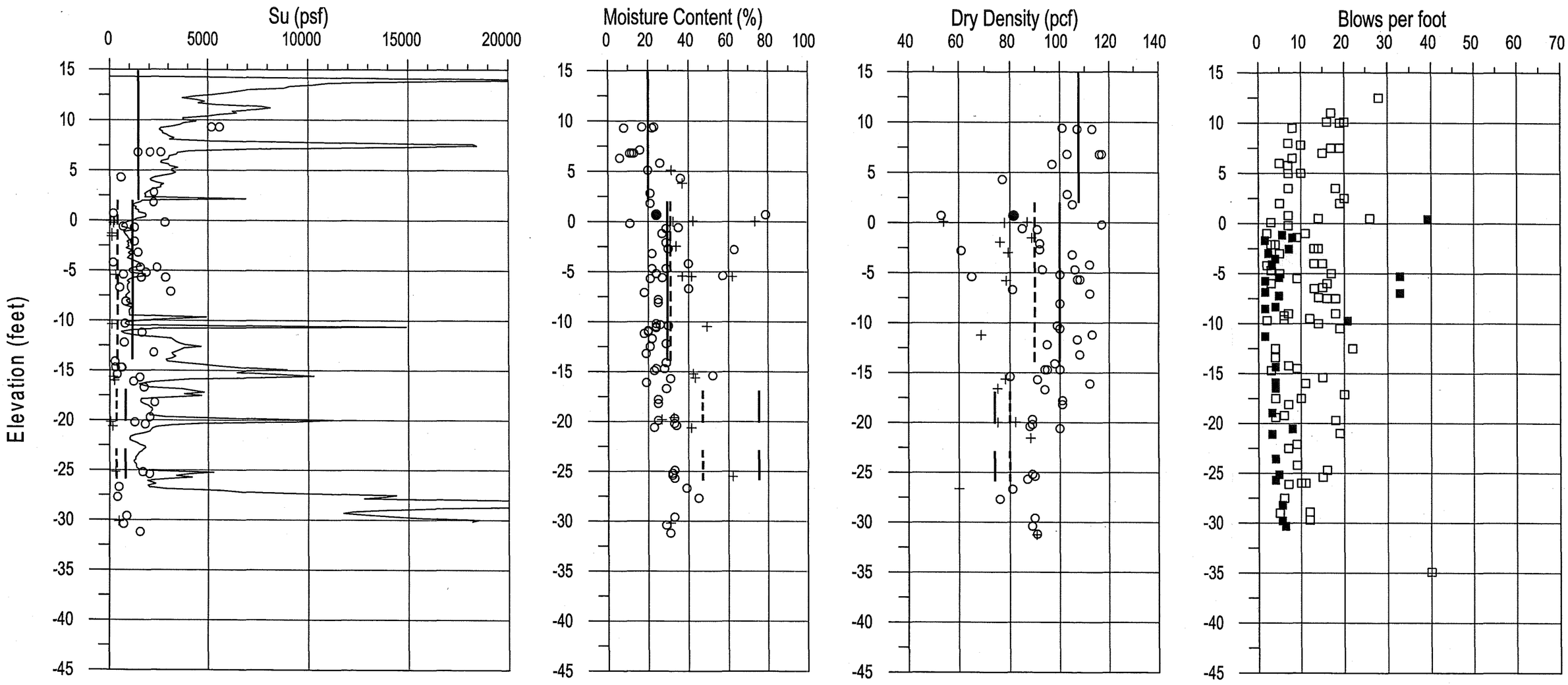
LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\244\index10.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 10	Figure 13
6/1/02	URS		



Data from the following borings and CPTs:

GRI-EB2	GSB-4	RREMB-5	EB-19
GRI-EB3	GSB-5	RREMB-6	EB-20
GRI-EB4	GSBOLD-1	SPRRB-R2	WB-01
GRI-EB5	GSBOLD-2	SPRRB-R3	WB-02
GRIV-21	GSBOLD-3	SPRRB-R4	CB-01
GROCS-1	GSBOLD-4	SPRRB-R5	EC-01
GSB-1	GSBOLD-5	SPRRB-R6	
GSB-2	GSBOLD-6	SPRRB-R7	
GSB-3	RREMB-4	EB-18	

Levee Fill and Foundation Soil

- + Fat Clay
- ○ Lean Clay
- □ All Soil Types

Channel Sediments

- + Fat Clay
- ● Lean Clay
- ■ All Soil Types

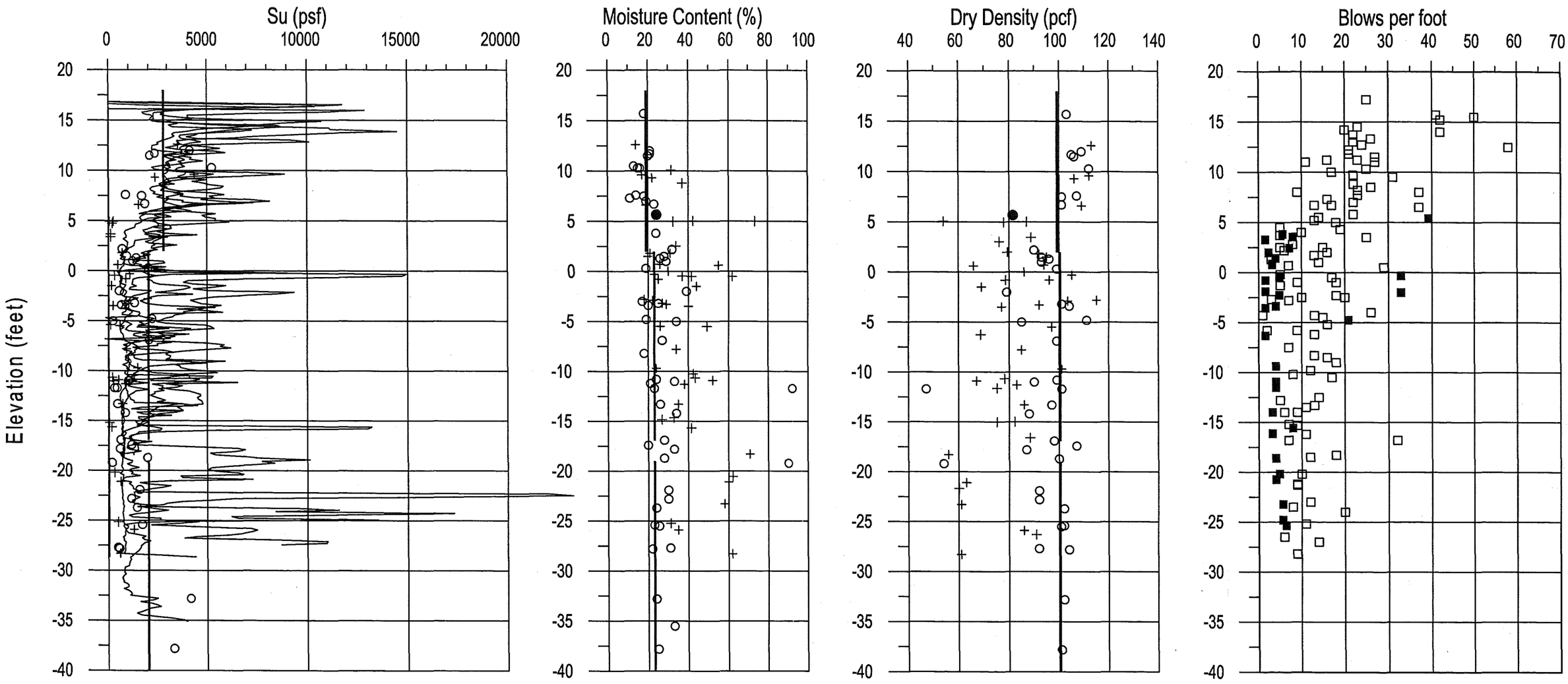
LEGEND:

| SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

¦ SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\244\index11.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 11	Figure 14
6/1/02	URS		



Data from the following borings and CPTs:

GRI-EB10	GRIV-19	WB-02	WC-01
GRI-EB6	EB-02	WB-18	WC-02
GRI-EB7	EB-03	WB-19	
GRI-EB8	EB-04	WB-20	
GRI-EB9	EB-07	EC-02	
GRIV-17	EB-21	EC-03	
GRIV-18	EB-22	EC-04	

Levee Fill and Foundation Soil

+ Fat Clay

○ Lean Clay

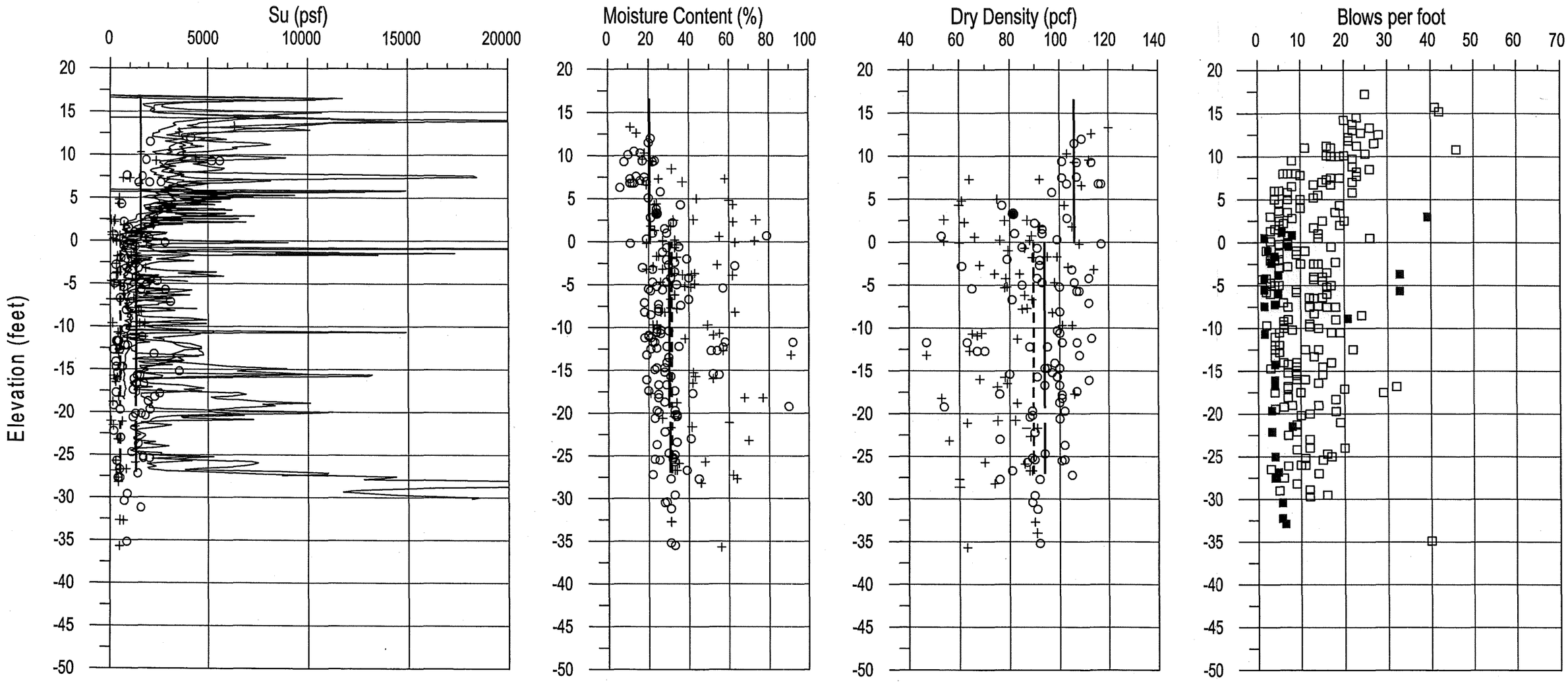
□ All Soil Types

LEGEND:

| SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 12	Figure 15
6/1/02	URS		

Q:\cadd2\01files\95-00112030.00\244\index12.dwg



Data from the following borings and CPTs:

BSP-1	GRI-EB6	GRIV-21	GSBOLD-2	RREMB-6	EB-04	WB-18	WC-01
BSP-2	GRI-EB7	GROCS-1	GSBOLD-3	SPRRB-R1	EB-05	WB-19	WC-02
BSP-3	GRI-EB8	GSB-1	GSBOLD-4	SPRRB-R2	EB-06	WB-20	
GJJ-6	GRI-EB9	GSB-2	GSBOLD-5	SPRRB-R3	EB-07	CB-01	
GRI-EB1	GRIM-6	GSB-2A	GSBOLD-6	SPRRB-R4	EB-18	CB-14	
GRI-EB10	GRIM-7	GSB-2B	RREMB-1	SPRRB-R5	EB-19	CB-15	
GRI-EB2	GRIV-17	GSB-3	RREMB-2	SPRRB-R6	EB-20	EC-01	
GRI-EB3	GRIV-18	GSB-4	RREMB-3	SPRRB-R7	EB-27	EC-02	
GRI-EB4	GRIV-19	GSB-5	RREMB-4	EB-02	WB-01	EC-03	
GRI-EB5	GRIV-20	GSBOLD-1	RREMB-5	EB-03	WB-02	EC-04	

Levee Fill and Foundation Soil

+ Fat Clay
○ Lean Clay
□ All Soil Types

Channel Sediments

+ Fat Clay
● Lean Clay
■ All Soil Types

LEGEND:

SELECTED DESIGN PARAMETER FOR LEVEE FILL AND FOUNDATION SOILS.

SELECTED DESIGN PARAMETER FOR CHANNEL SEDIMENTS.

Q:\cadd2\01files\95-00112030.00\244\index13

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	INDEX PROPERTY PLOTS FOR CROSS SECTION NO. 13	Figure 16
6/1/02	URS		

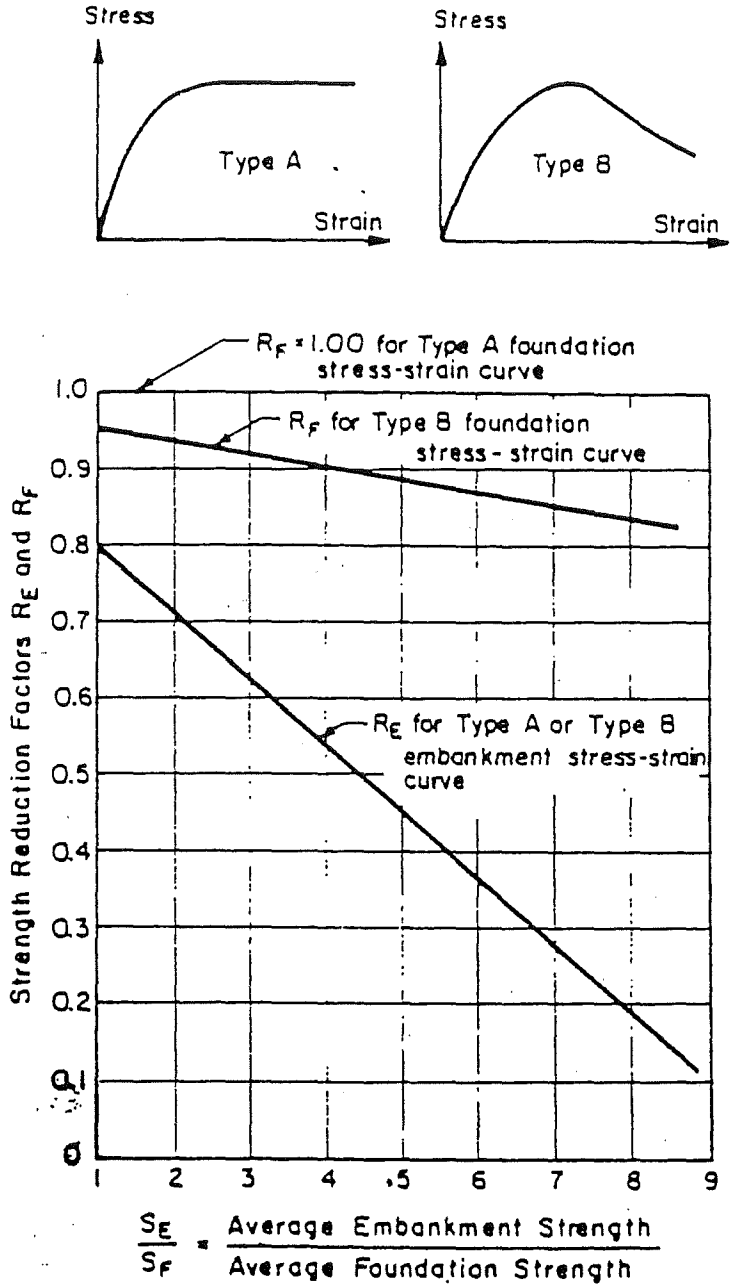
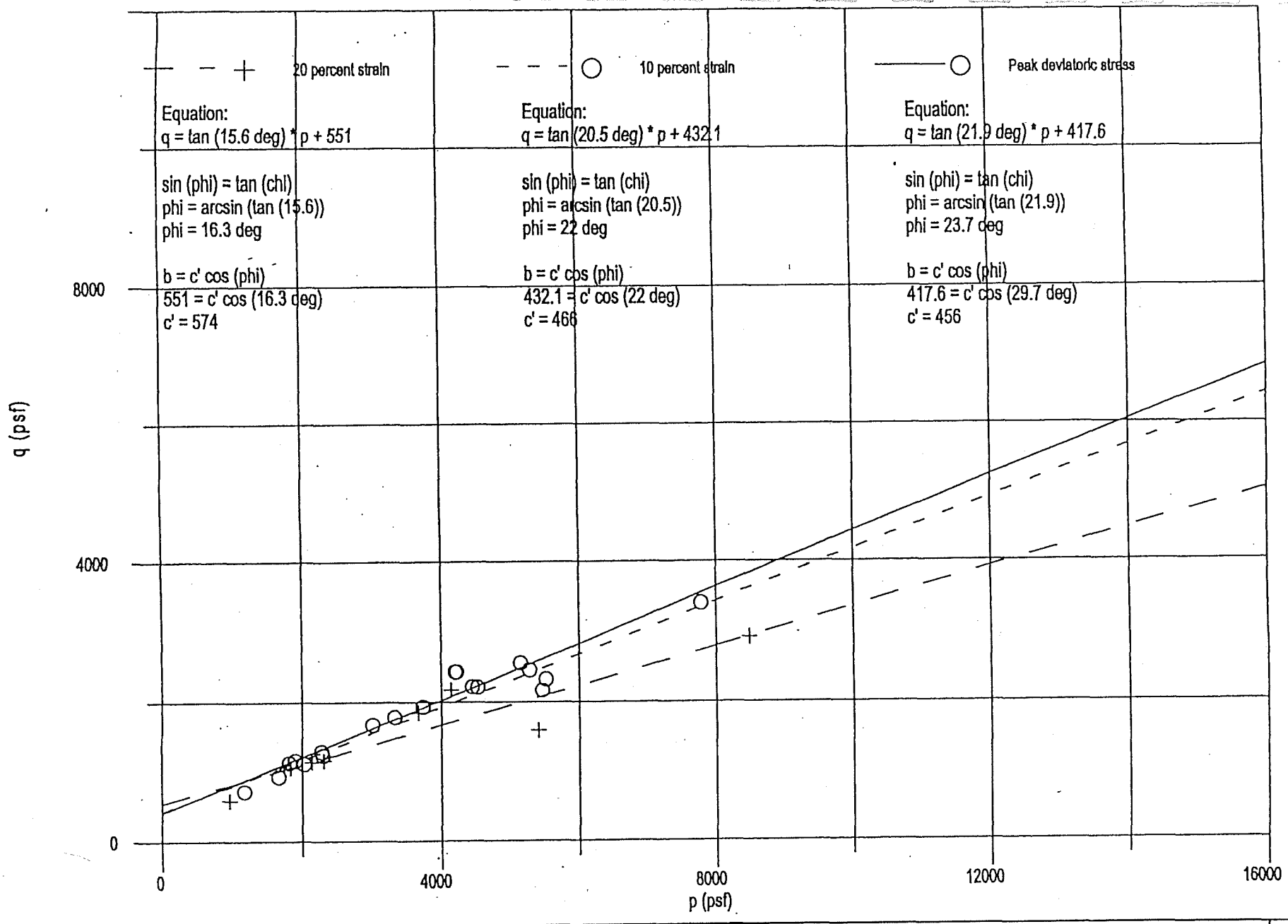


Fig. 3 CORRECTION FACTORS R_E AND R_F TO ACCOUNT FOR PROGRESSIVE FAILURE IN EMBANKMENTS ON SOFT CLAY FOUNDATIONS.
(after Chirapuntu and Duncan, 1975)

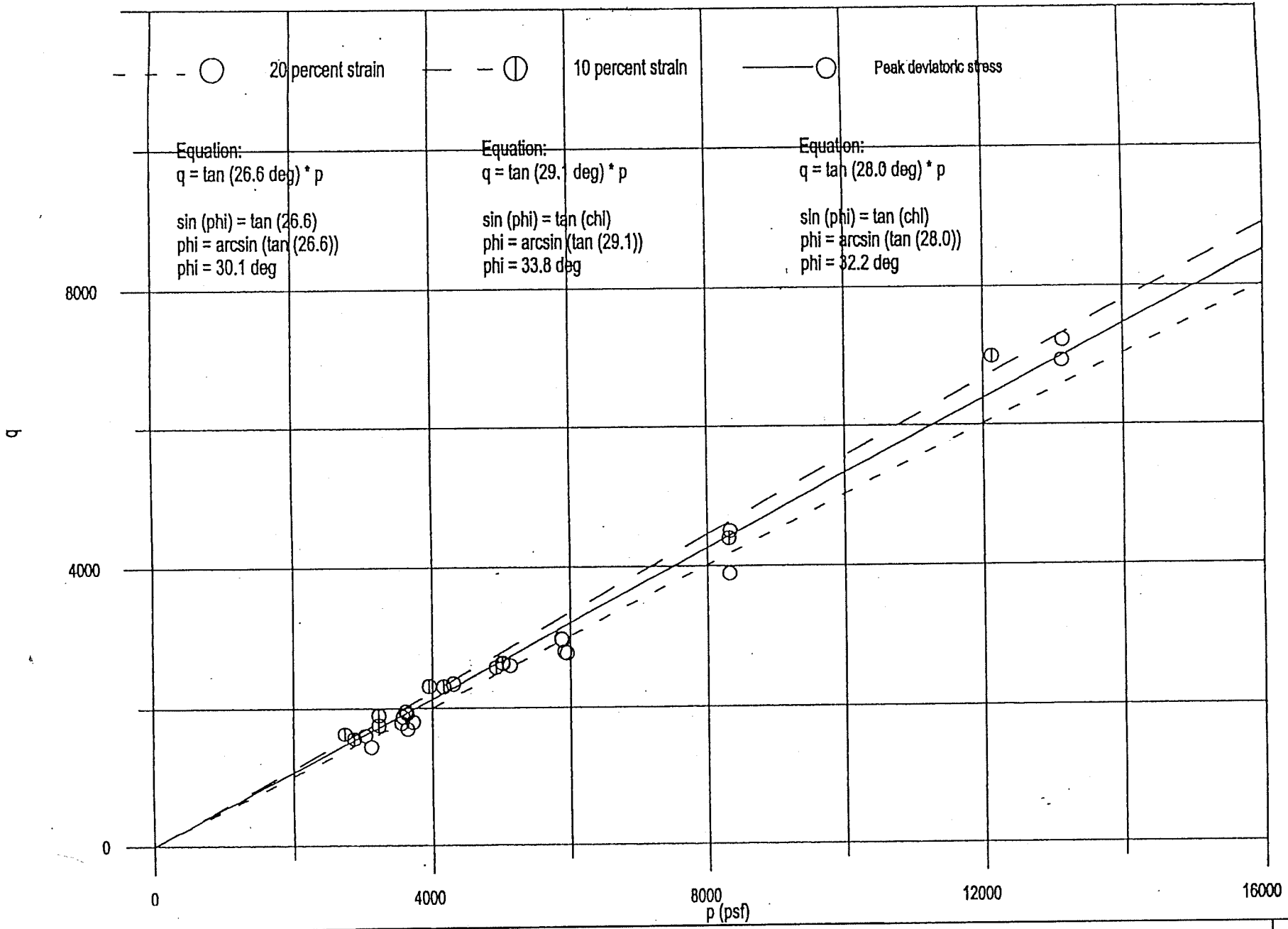
Q:\CADD2\01FILES\95-00112030.00\GUADFIG017.DWG

Project No. 95-00112030.00 6/1/02	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	EMBANKMENT STRENGTH REDUCTION FACTOR FOR SOFT CLAY FOUNDATIONS	Figure 17
URS			



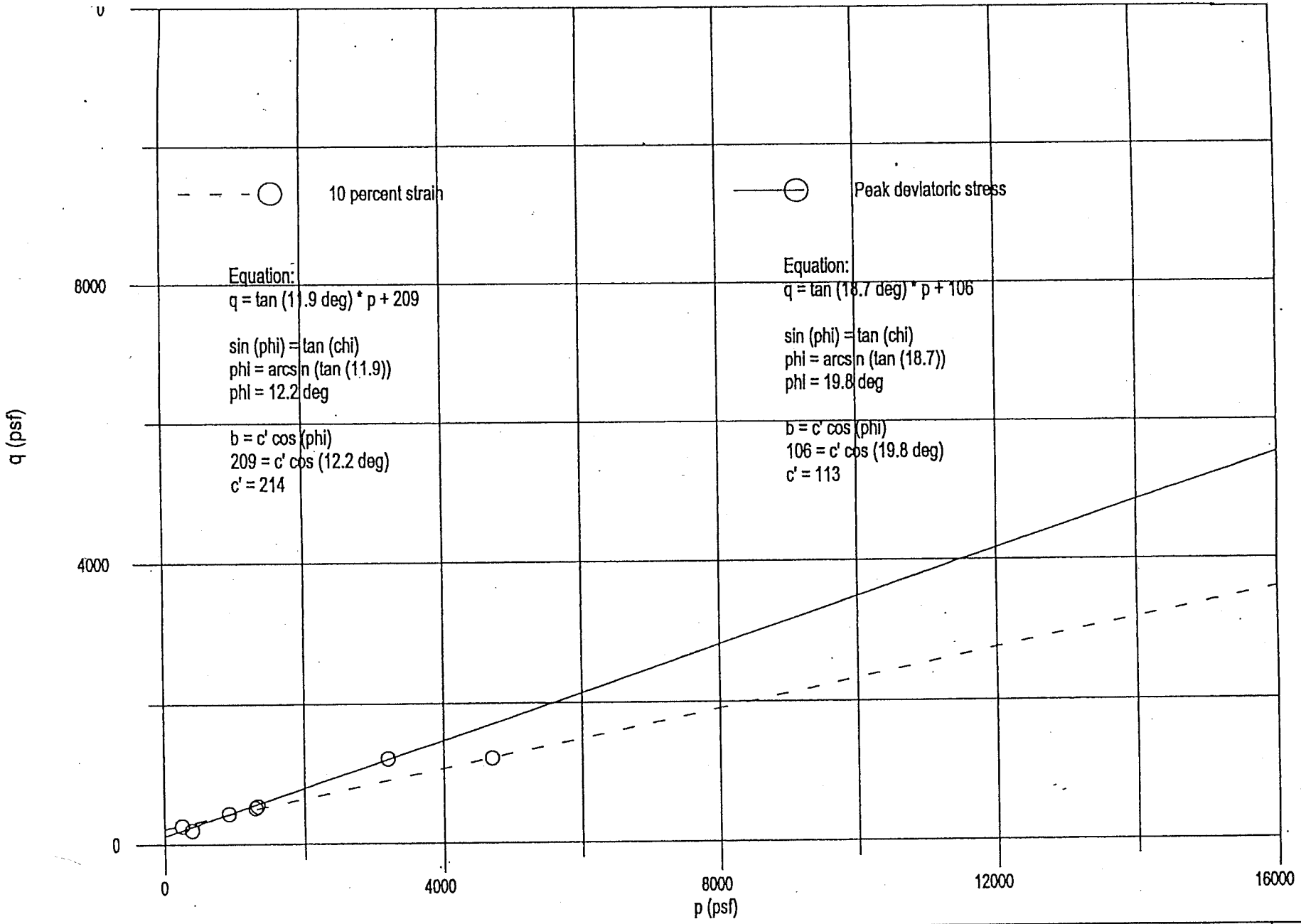
Q:\CADD2\01FILES\95-00112030.00\244\GUADFIG1B.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	**Shear Strength Parameters For Lean Clay Fill with Dry Density less than 110 pcf**	Figure 18
6/1/02			
URS			



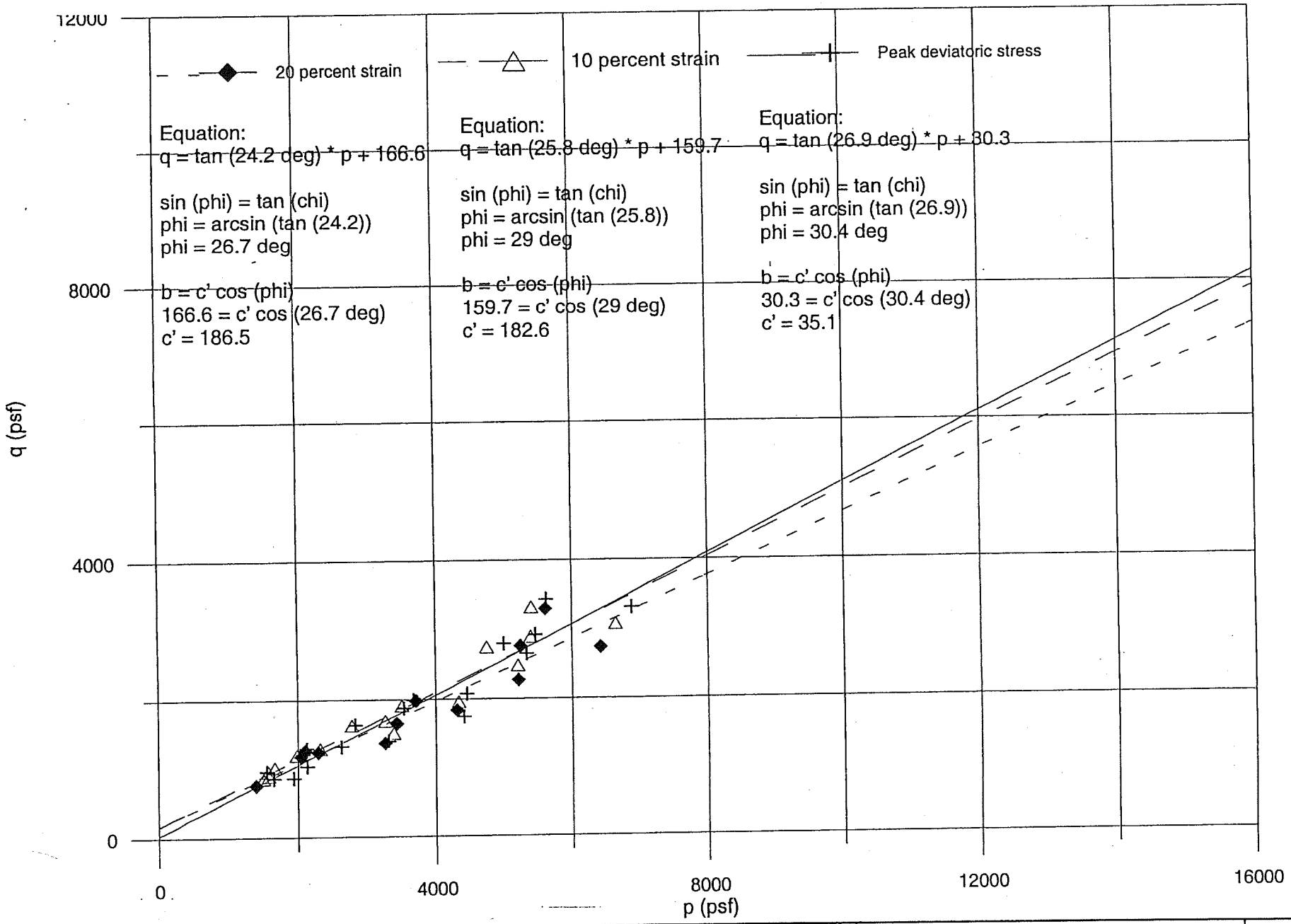
Q:\CADD2\01FILES\95-00112030.00\244\GUADFIG19.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	**Shear Strength Parameters For Lean Clay Fill with Dry Density greater than 110 pcf**	Figure 19
6/1/02			
URS			



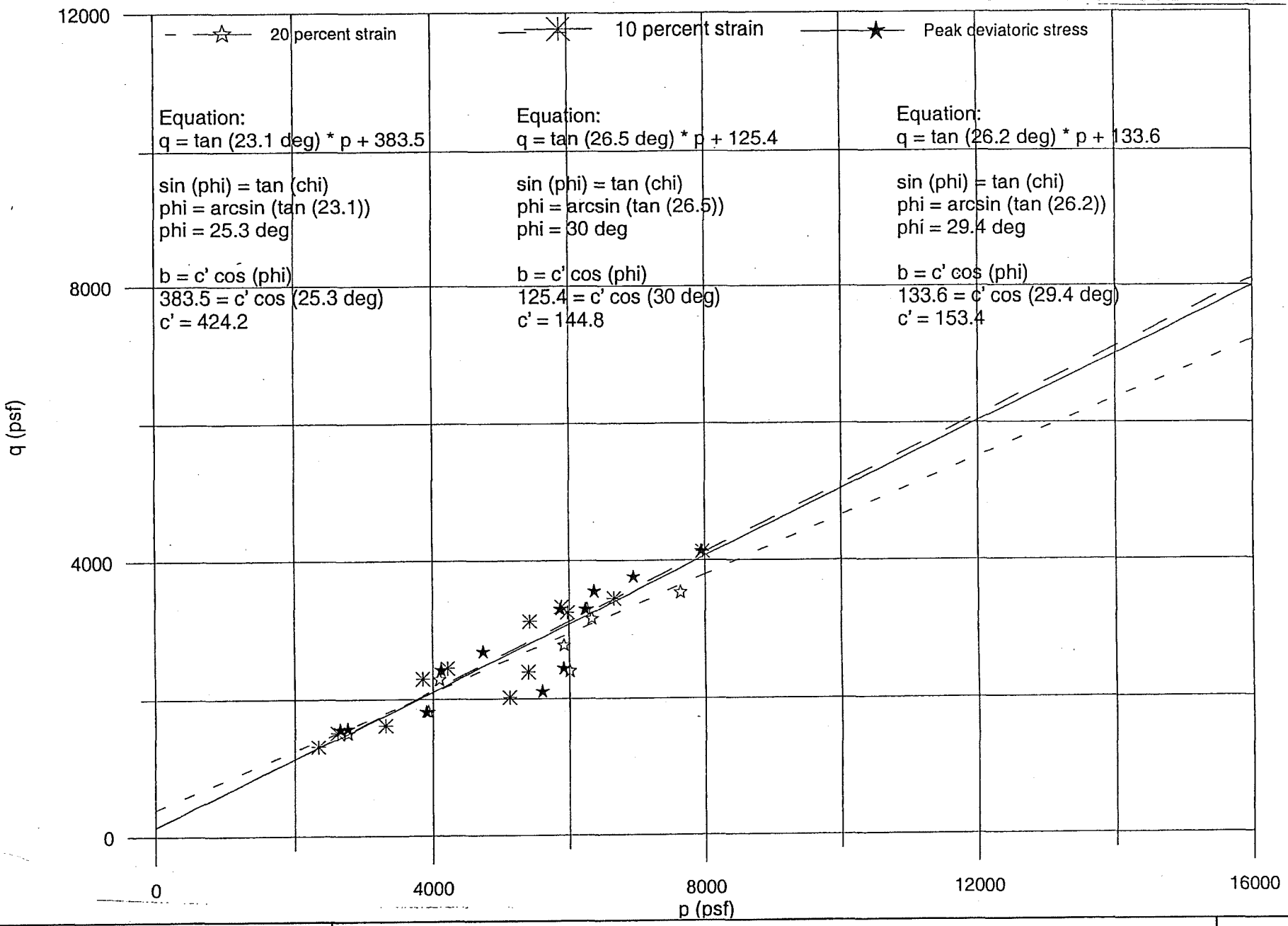
Q:\CADD2\01FILES\95-00112030.00\244\GUADFIG20.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	**Shear Strength Parameters For Fat Clay with Dry Density less than 80 pcf**	Figure 20
6/1/02			
URS			



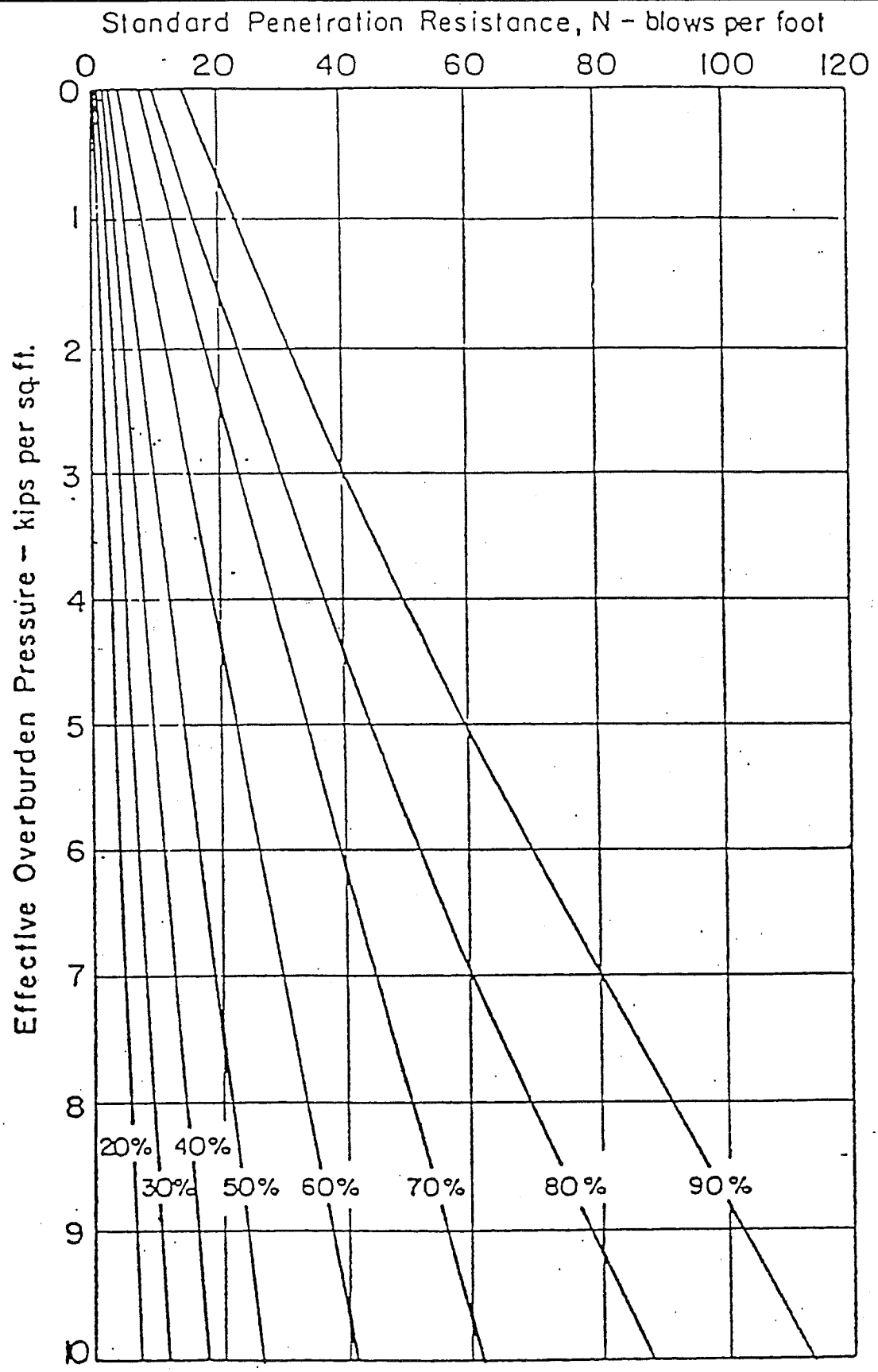
Q:\CADD2\01FILES\95-00112030.00\244\GUADFIG21.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	**Shear Strength Parameters For Lean Clay with Dry Density between 80 pcf and 100 pcf**	Figure 21
6/1/02			
URS			



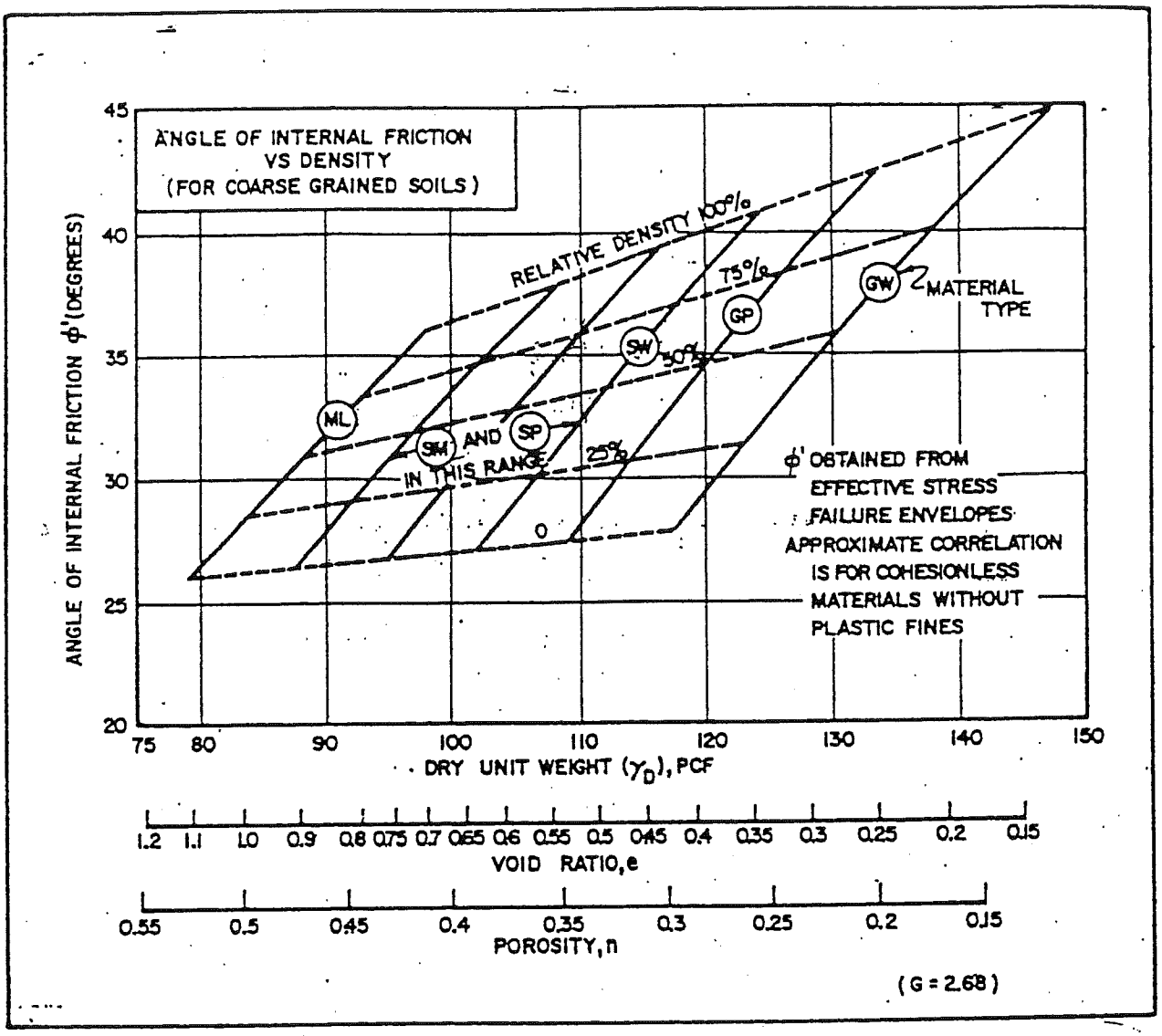
Q:\CADD2\01FILES\95-00112030.00\244\GUADFIG22.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	**Shear Strength Parameters For Lean Clay with Dry Density between 100 pcf and 110 pcf**	Figure 22
6/1/02			
URS			



Q:\CADD2\01FILES\95-00112030.00\GUADFIG023.DWG

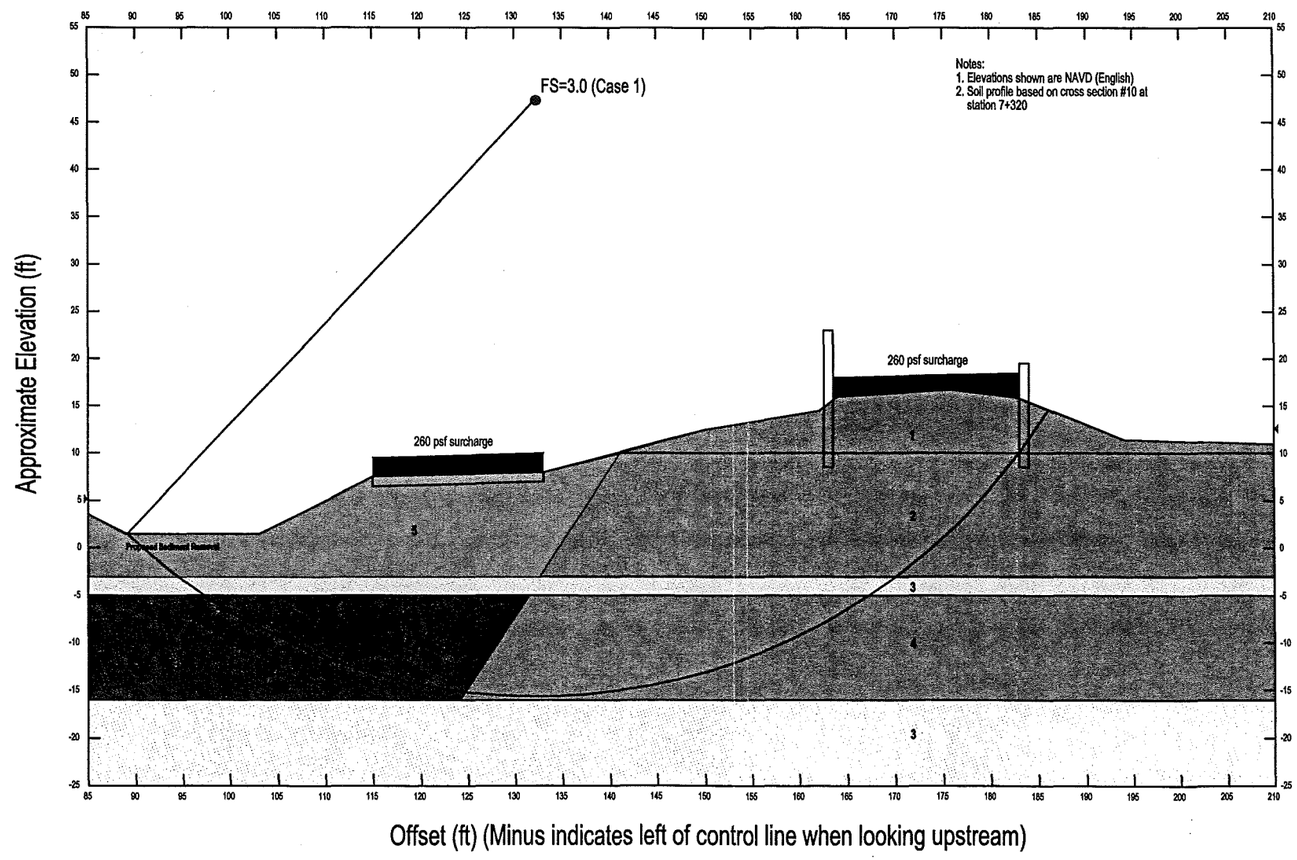
Project No. 95-00112030.00 6/1/02	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	**Relationship between Standard Penetration Resistance, Relative Density and Effective Overburden Pressure** (AFTER GIBBS AND HOLTZ, 1957)	Figure 23
URS			



Q:\CADD2\01FILES\95-00112030.00\244\GUADFIG24.DWG

Project No. 95-00112030.00 6/1/02	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Correlation of Strength Characteristics for Granular Soils (NAVFAC, 1986)	Figure 24
URS			

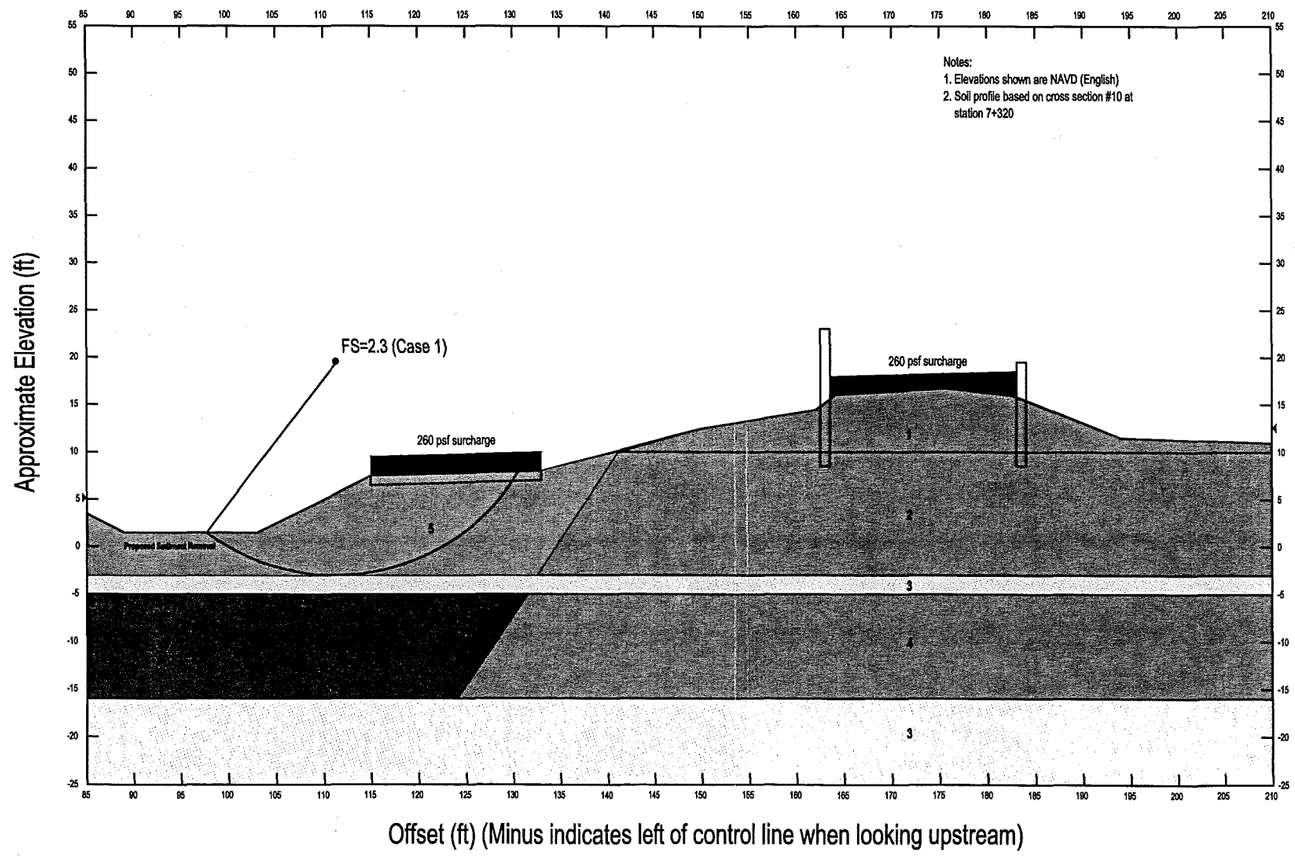
Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 5				33	0		
		Figure 13	110	18	130			3000	
1	Lean CLAY (FILL)		110	18	130	33	0	1880	Selected
		Table 5				24	450		
		Figure 13	90	30	117			1000	
2	Fat CLAY		90	30	117	24	450	1000	Selected
						33	0		Estimated from N = 10, Sv' = 3.3 ksf, Dr = 50%
3	Silty SAND				135	33	0		Selected
		Table 5				30	35		
		Figure 13	98	30	127			1000	
4	Lean CLAY		98	30	127	30	35	1000	Selected
5	Fat CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	87	36	118	20	110	400	Selected
6	Lean CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	90	30	118	30	35	425	Selected



q:\cadd2\01files\95-00112030.00\244\Fig25.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS DEPRESSED MAINTENANCE ROAD STATION 7+200 WEST LEVEE	Figure 25
6/1/02	URS		

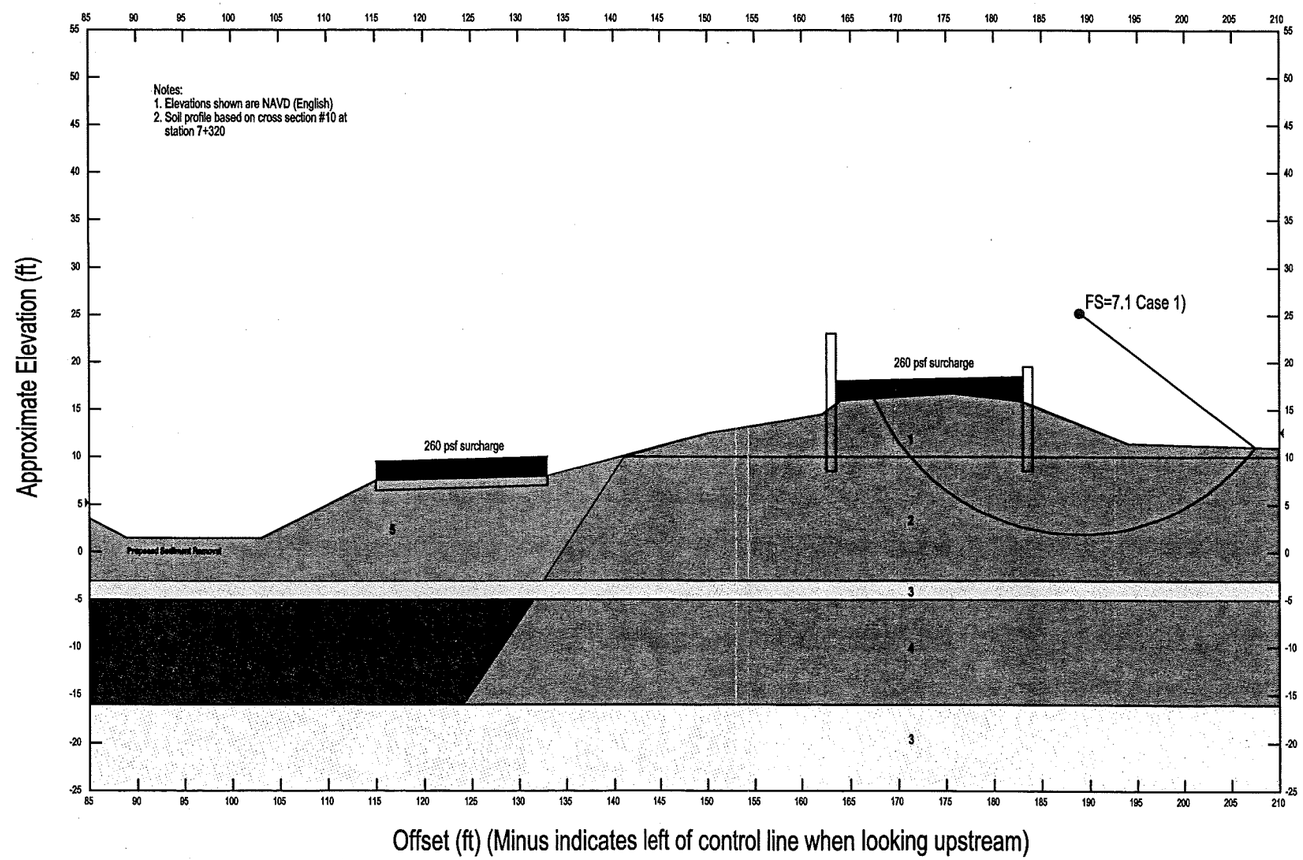
Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 5				33	0		
		Figure 13	110	18	130			3000	
1	Lean CLAY (FILL)		110	18	130	33	0	1880	Selected
		Table 5				24	450		
		Figure 13	90	30	117			1000	
2	Fat CLAY		90	30	117	24	450	1000	Selected
						33	0		Estimated from N = 10,
3	Silty SAND				135	33	0		Sv' = 3.3 ksf, Dr = 50% Selected
		Table 5				30	35		
		Figure 13	98	30	127			1000	
4	Lean CLAY		98	30	127	30	35	1000	Selected
5	Fat CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	87	36	118	20	110	400	Selected
6	Lean CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	90	30	118	30	35	425	Selected



q:\cadd2\01files\95-00112030.00\244\Fig26.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS DEPRESSED MAINTENANCE ROAD STATION 7+200 WEST LEVEE	Figure 26
6/1/02	URS		

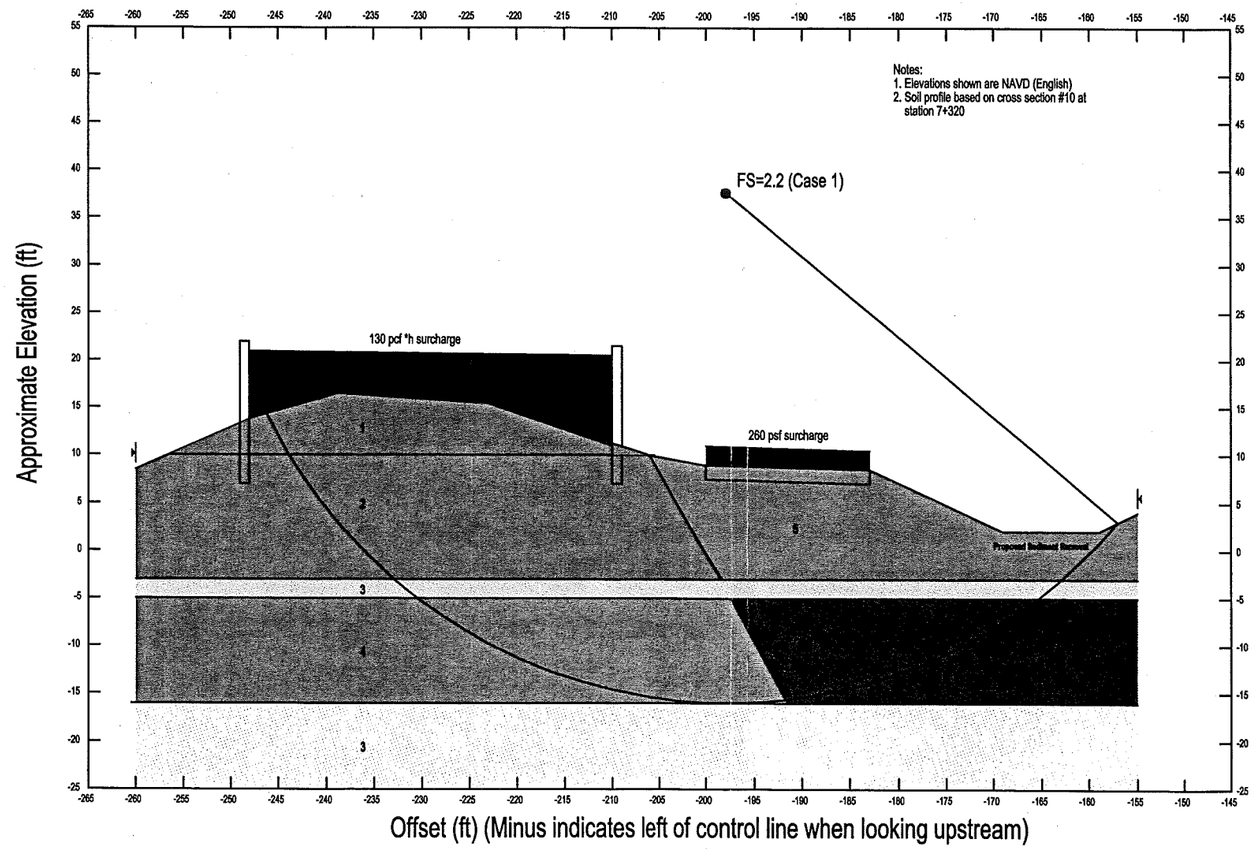
Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 5 Figure 13	110 110	18 18	130 130	33 33	0 0	3000 1880	Selected
2	Fat CLAY	Table 5 Figure 13	90 90	30 30	117 117	24 24	450 450	1000 1000	Selected
3	Silty SAND				135	33 33	0 0		Estimated from N = 10, Sv' = 3.3 ksf, Dr = 50% Selected
4	Lean CLAY	Table 5 Figure 13	98 98	30 30	127 127	30 30	35 35	1000 1000	Selected
5	Fat CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	87	36	118	20	110	400	Selected
6	Lean CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	90	30	118	30	35	425	Selected



q:\cadd2\01files\95-00112030.00\244\Fig27.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS DEPRESSED MAINTENANCE ROAD STATION 7+200 WEST LEVEE	Figure 27
6/1/02	URS		

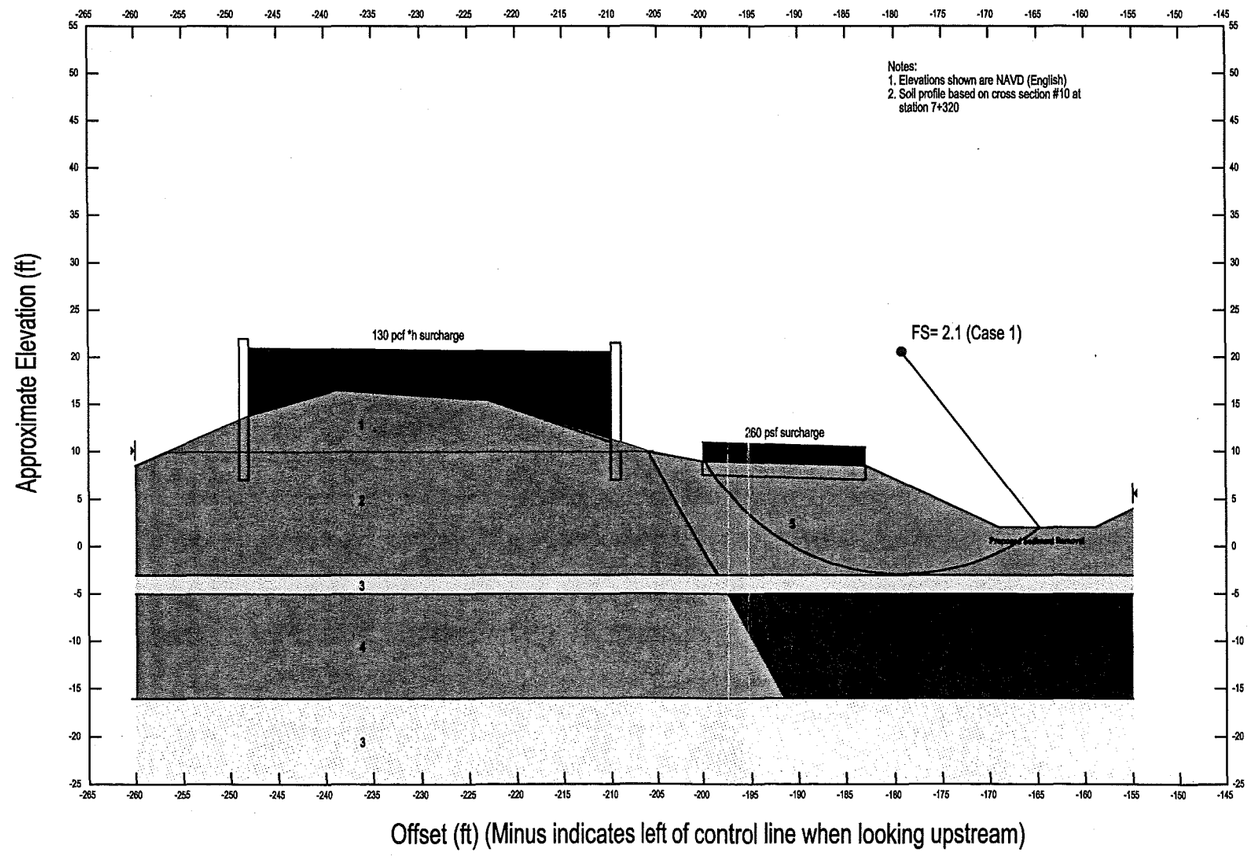
Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 5 Figure 13	110 110	18 18	130 130	33 33	0 0	3000 1880	Selected
2	Fat CLAY	Table 5 Figure 13	90 90	30 30	117 117	24 24	450 450	1000 1000	Selected
3	Silty SAND				135	33 33	0 0		Estimated from N = 10, Sv' = 3.3 ksf, Dr = 50% Selected
4	Lean CLAY	Table 5 Figure 13	98 98	30 30	127 127	30 30	35 35	1000 1000	Selected
5	Fat CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	87	36	118	20	110	400	Selected
6	Lean CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	90	30	118	30	35	425	Selected



q:\cadd2\01files\95-00112030.00\244\Fig28.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS DEPRESSED MAINTENANCE ROAD STATION 7+200 EAST LEVEE	Figure 28
6/1/02	URS		

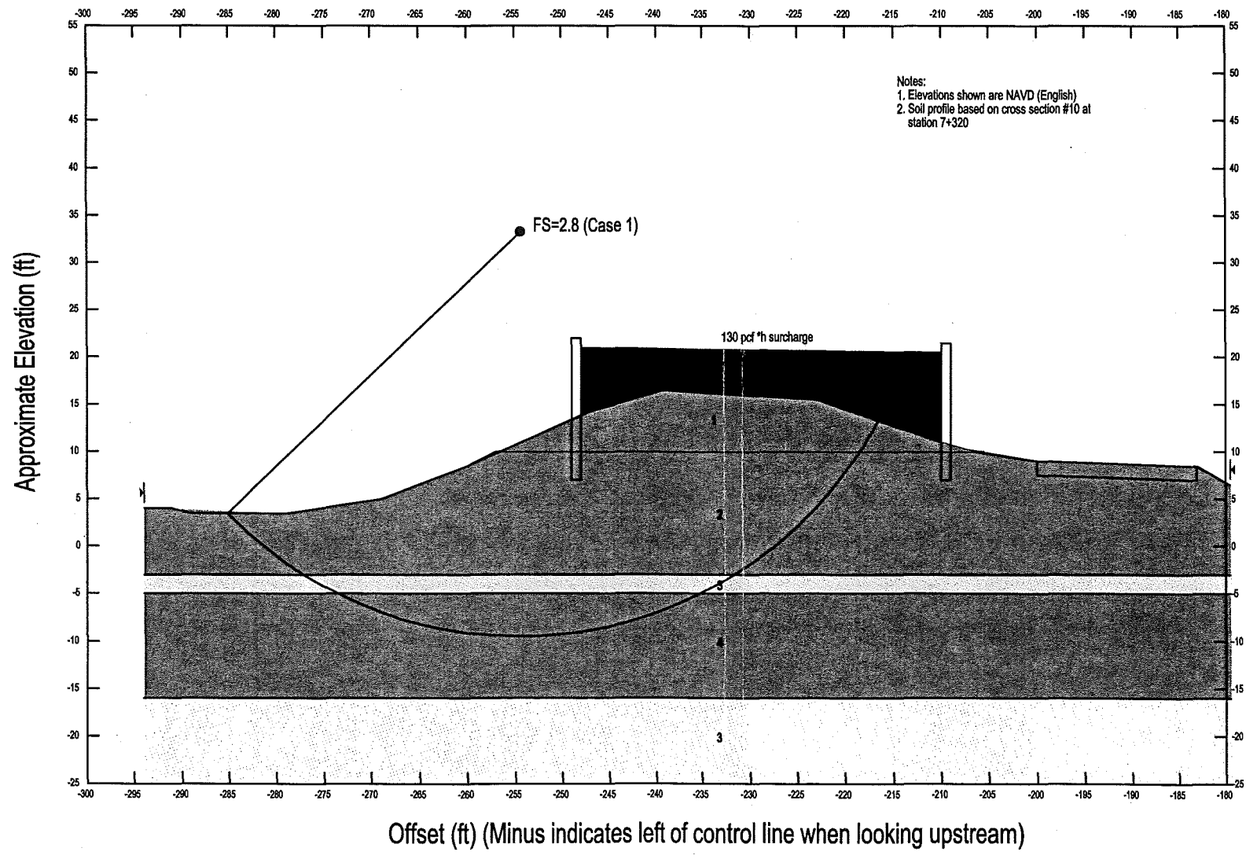
Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 5 Figure 13	110 110	18 18	130 130	33 33	0 0	3000 1880	Selected
2	Fat CLAY	Table 5 Figure 13	90 90	30 30	117 117	24 24	450 450	1000 1000	Selected
3	Silty SAND				135	33 33	0 0		Estimated from N = 10, Sv' = 3.3 ksf, Dr = 50% Selected
4	Lean CLAY	Table 5 Figure 13	98 98	30 30	127 127	30 30	35 35	1000 1000	Selected
5	Fat CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	87	36	118	20	110	400	Selected
6	Lean CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	90	30	118	30	35	425	Selected



q:\cadd2\01files\95-00112030.00\244\Fig29.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS DEPRESSED MAINTENANCE ROAD STATION 7+200 EAST LEVEE	Figure 29
6/1/02	URS		

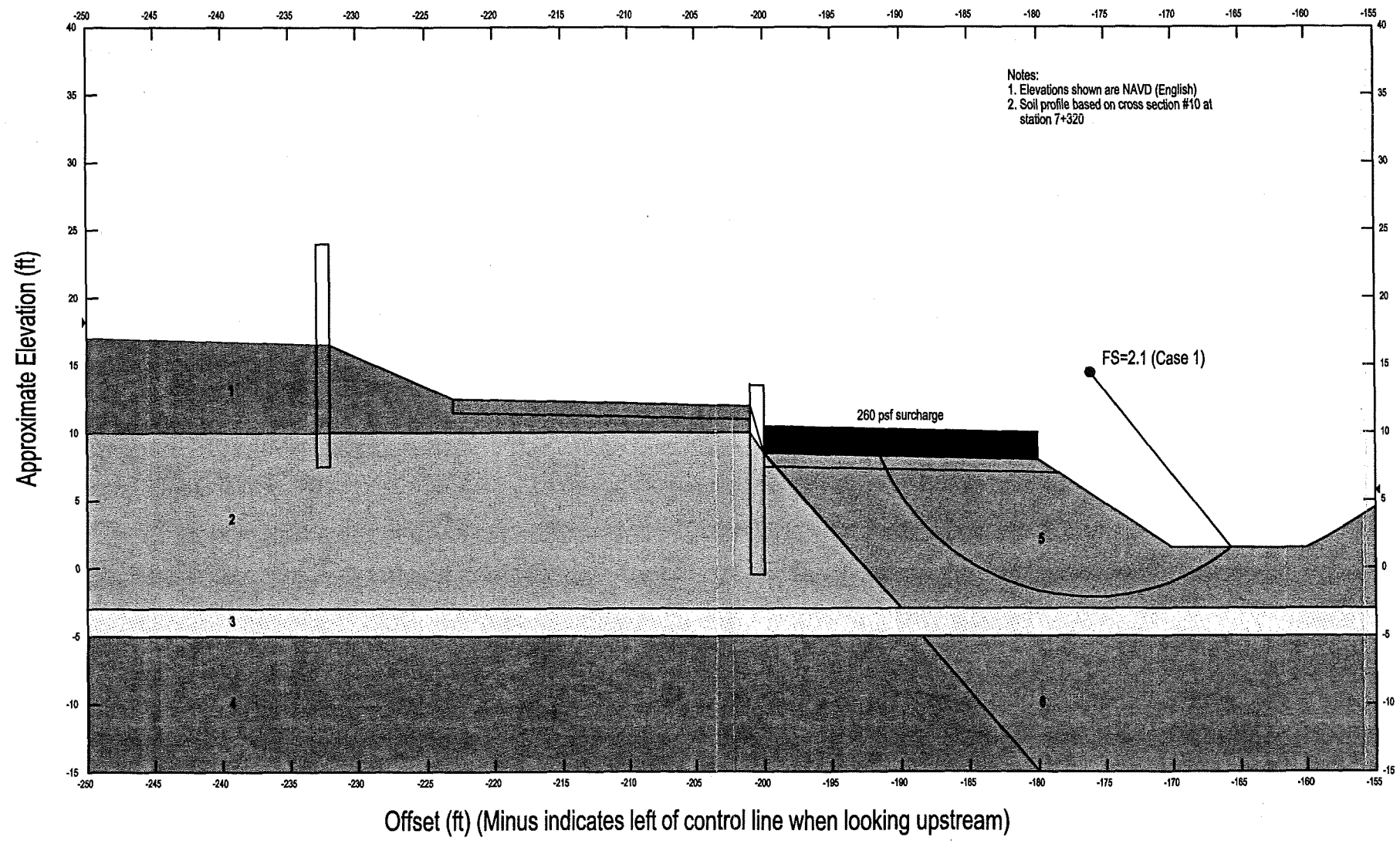
Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
		Table 5				33	0		
		Figure 13	110	18	130			3000	
1	Lean CLAY (FILL)		110	18	130	33	0	1880	Selected
		Table 5				24	450		
		Figure 13	90	30	117			1000	
2	Fat CLAY		90	30	117	24	450	1000	Selected
						33	0		Estimated from N = 10, Sv' = 3.3 ksf, Dr = 50%
3	Silty SAND				135	33	0		Selected
		Table 5				30	35		
		Figure 13	98	30	127			1000	
4	Lean CLAY		98	30	127	30	35	1000	Selected
5	Fat CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	87	36	118	20	110	400	Selected
6	Lean CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	90	30	118	30	35	425	Selected



q:\cadd2\01files\95-00112030.00\244\Fig30.dwg

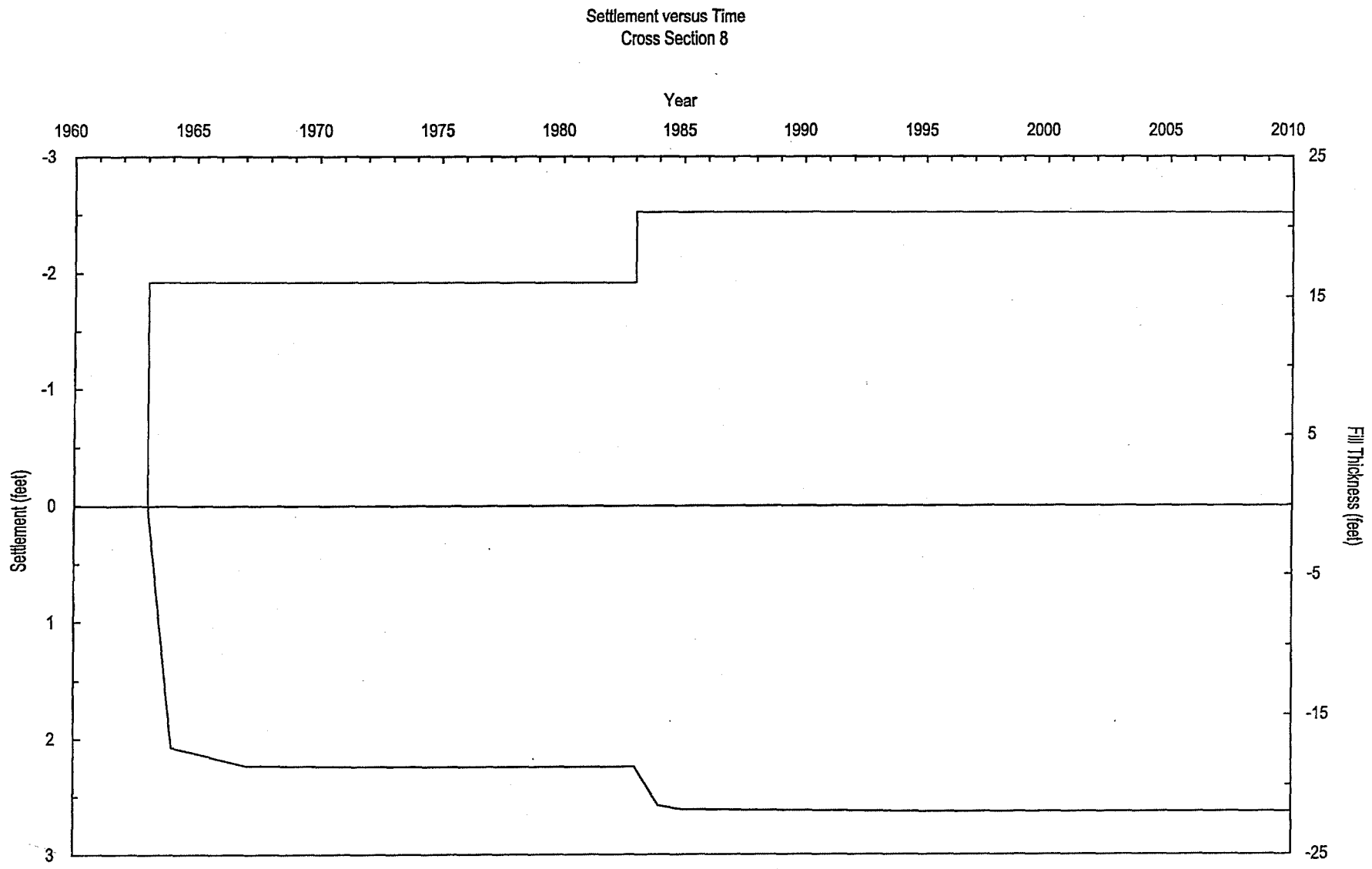
Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS DEPRESSED MAINTENANCE ROAD STATION 7+200 EAST LEVEE	Figure 30
6/1/02	URS		

Summary of Geotechnical Design Parameters									
Layer	Soil Type	Reference	Dry Density (pcf)	Moisture Content (%)	Total Unit Weight (pcf)	ϕ' (deg)	c' (psf)	Su (psf)	Remarks
1	Lean CLAY (FILL)	Table 5 Figure 13	110 110	18 18	130 130	33 33	0 0	3000 1880	Selected
2	Fat CLAY	Table 5 Figure 13	90 90	30 30	117 117	24 24	450 450	1000 1000	Selected
3	Silty SAND				135	33 33	0 0		Estimated from N = 10, Sv' = 3.3 ksf, Dr = 50% Selected
4	Lean CLAY	Table 5 Figure 13	98 98	30 30	127 127	30 30	35 35	1000 1000	Selected
5	Fat CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	87	36	118	20	110	400	Selected
6	Lean CLAY (Channel Sediments)	CB01, CB03, CB14, CB15	90	30	118	30	35	425	Selected



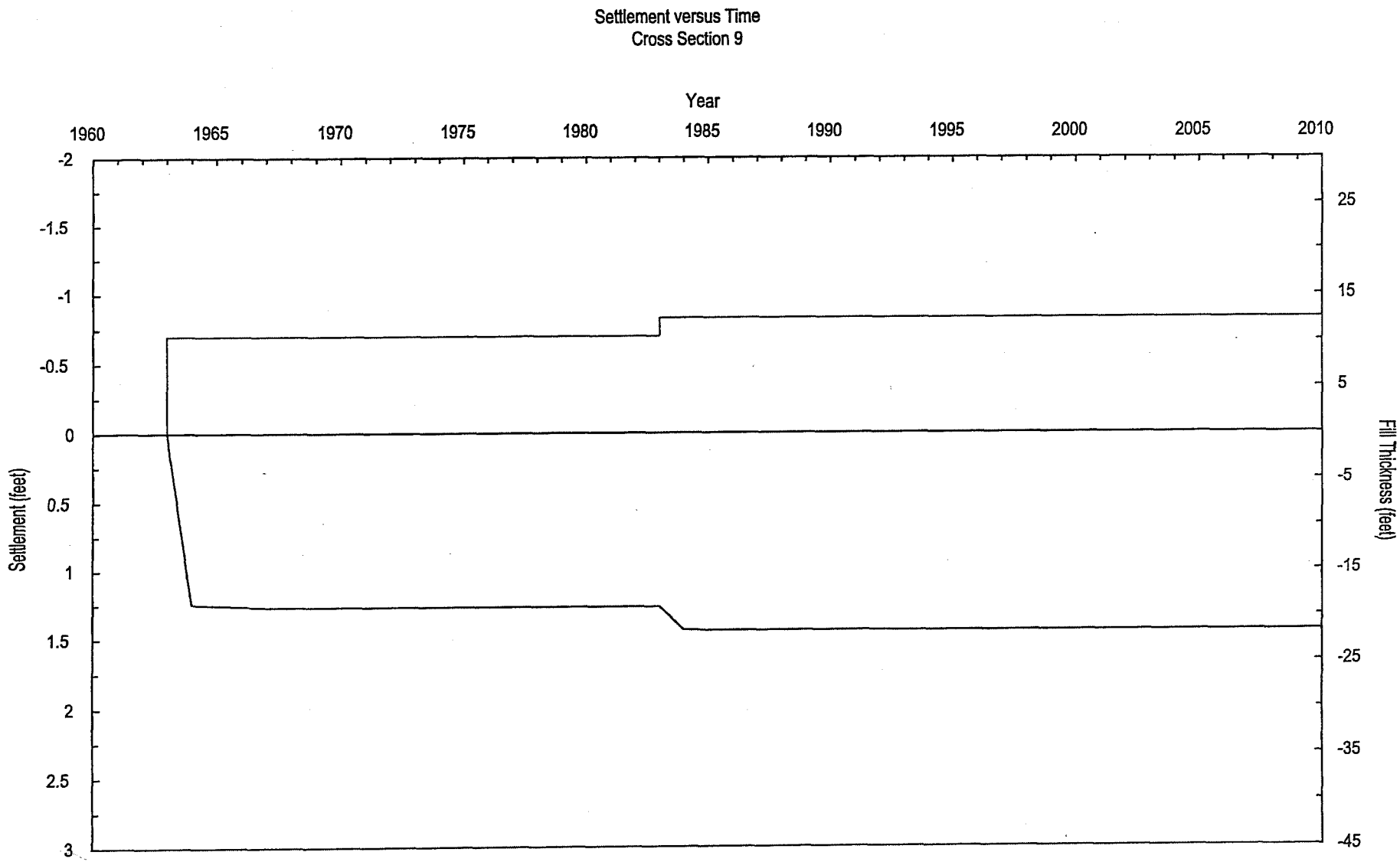
q:\cadd2\01files\95-00112030.00\244\Fig31.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STABILITY ANALYSIS DEPRESSED MAINTENANCE ROAD STATION 7+260 EAST LEVEE	Figure 31
6/1/02	URS		



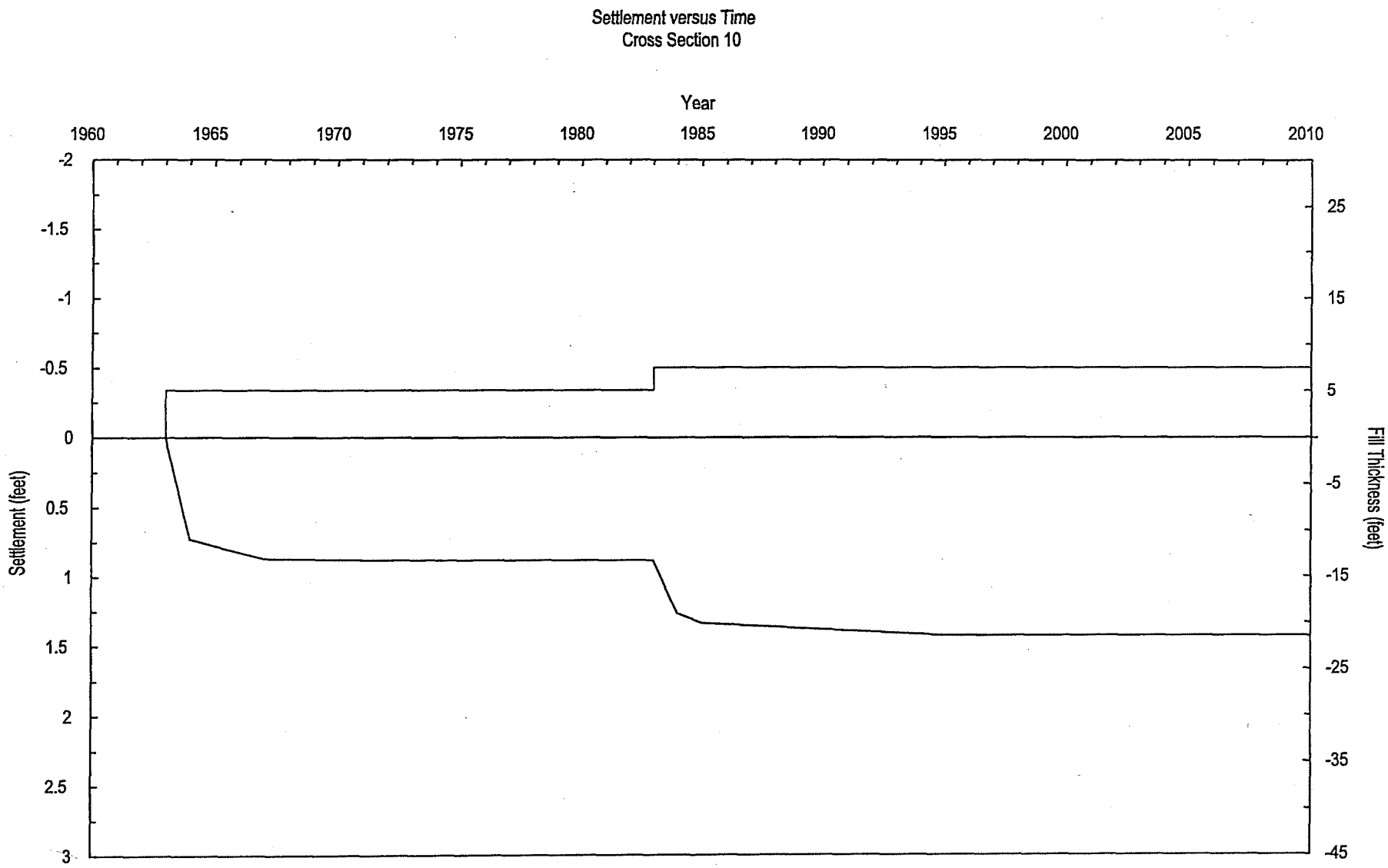
Q:\CADD2\01FILES\95-00112030.00\244\FIG32.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Estimated Time Rate of Settlement for Cross Section 8	Figure 32
6/1/02			
URS			



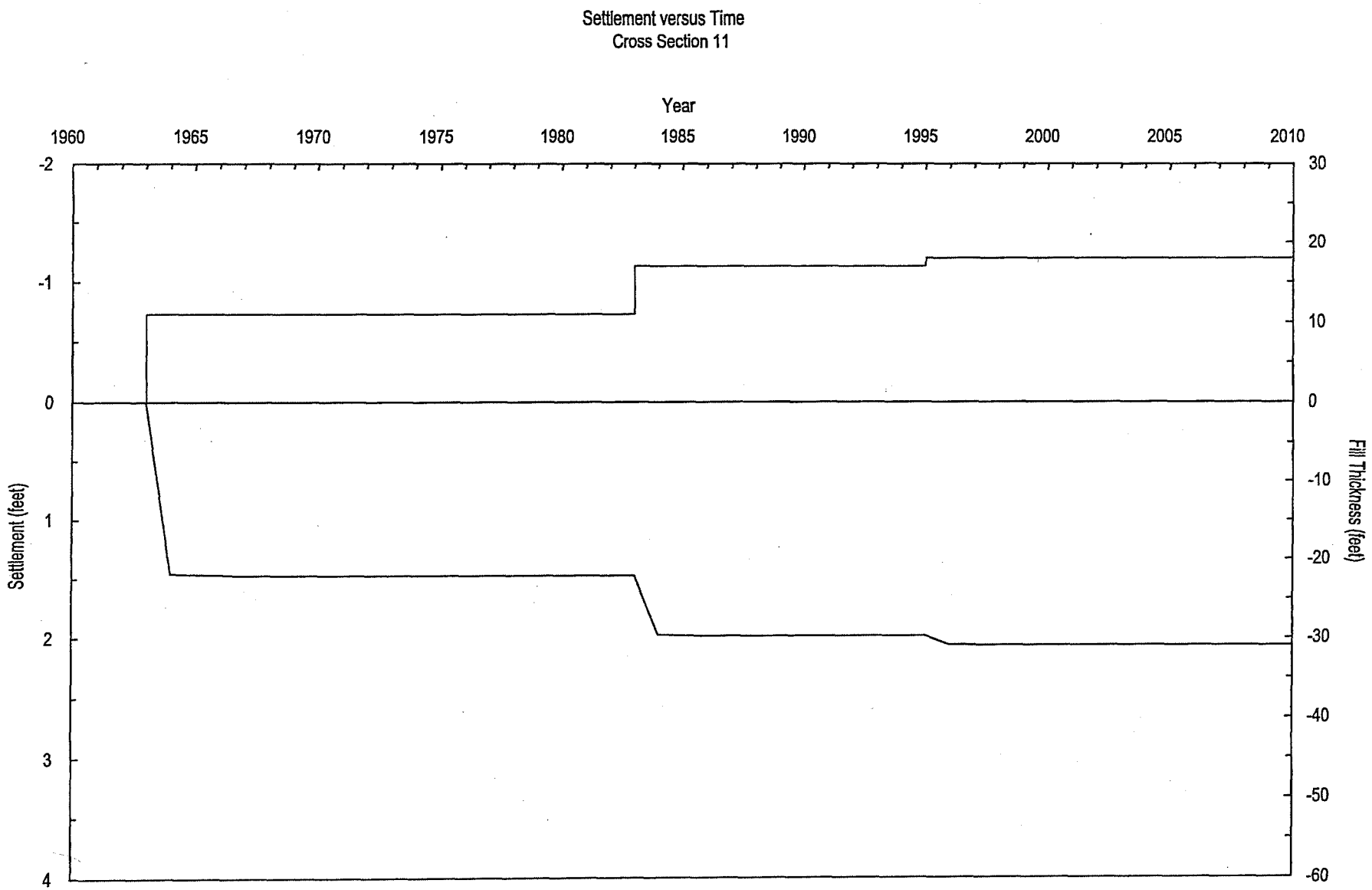
Q:\CADD2\01FILES\95-00112030.00\244\FIG33.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Estimated Time Rate of Settlement for Cross Section 9	Figure 33
6/1/02			
URS			



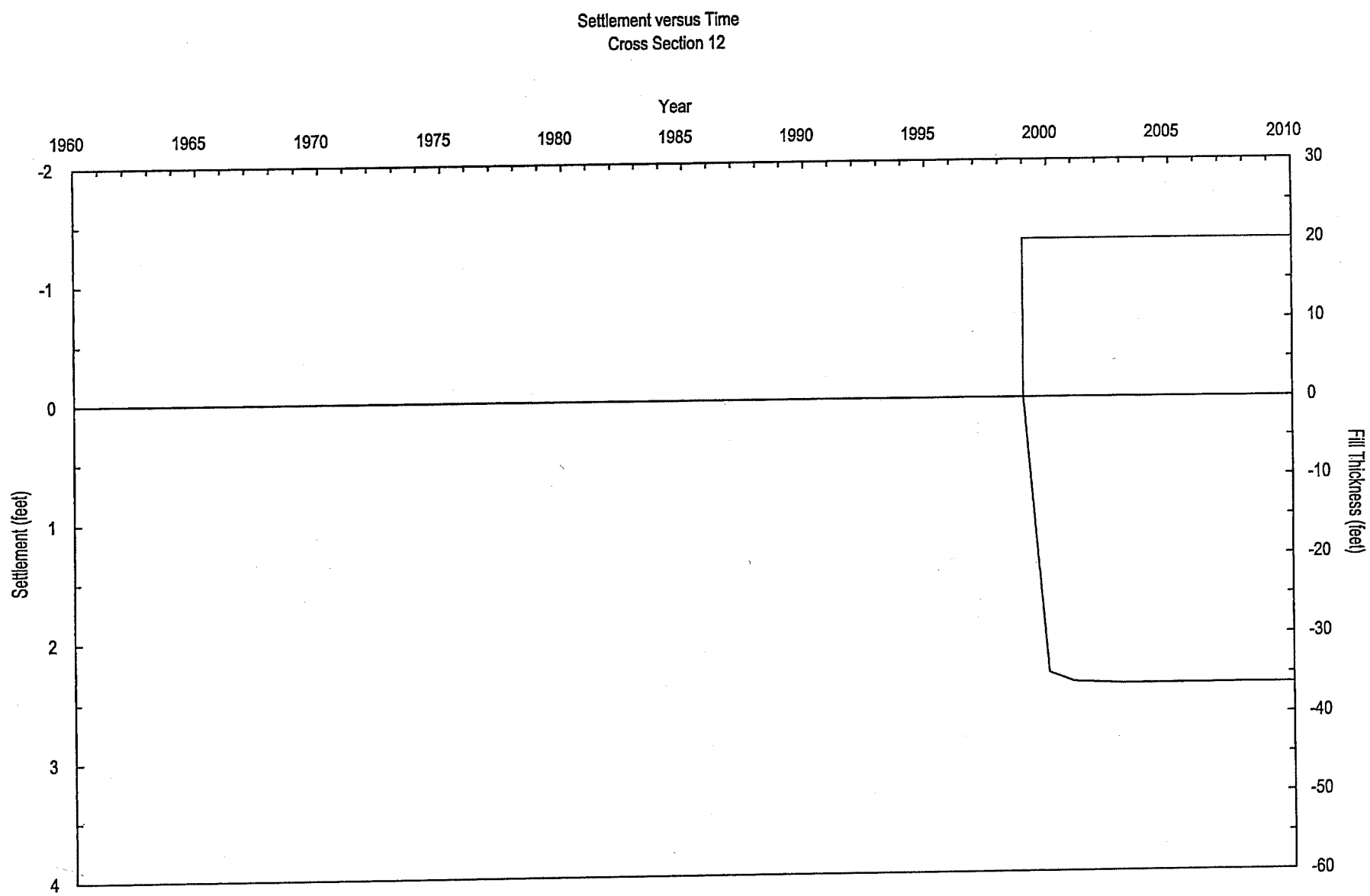
Q:\CADD2\01FILES\95-00112030.00\244\FIG34.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Estimated Time Rate of Settlement for Cross Section 10	Figure 34
6/1/02			
URS			



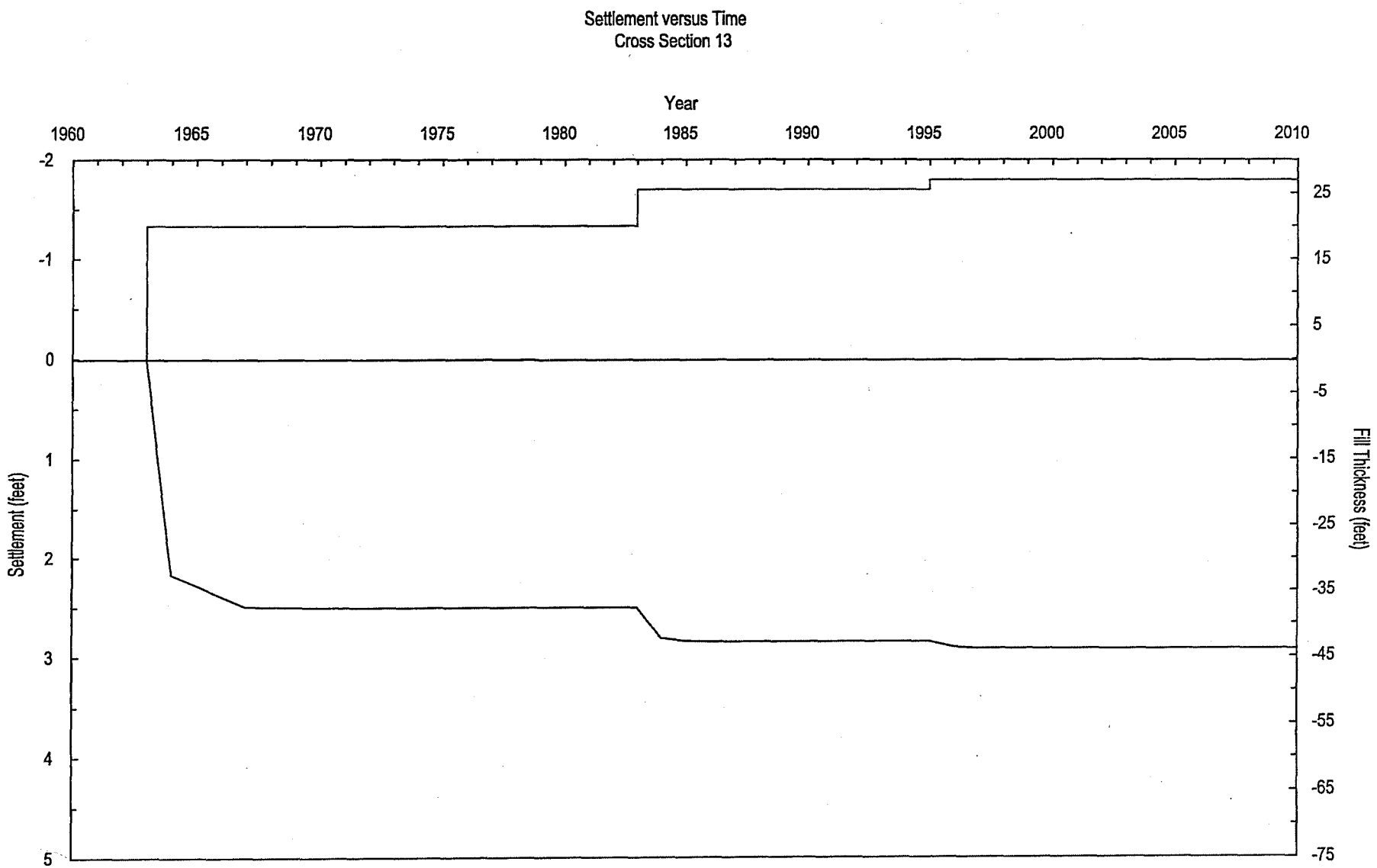
Q:\CADD2\01FILES\95-00112030.00\244\FIG35.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Estimated Time Rate of Settlement for Cross Section 11	Figure 35
6/1/02			
URS			



Q:\CADD2\01FILES\95-00112030.00\244\FIG36.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Estimated Time Rate of Settlement for Cross Section 12	Figure 36
6/1/02			
URS			



Q:\CADD2\01FILES\95-00112030.00\244\FIG37.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	Estimated Time Rate of Settlement for Cross Section 13	Figure 37
6/1/02			
URS			

Coefficient of Permeability k in cm per sec (log scale)

10^2 | 10^1 | 1.0 | 10^{-1} | 10^{-2} | 10^{-3} | 10^{-4} | 10^{-5} | 10^{-6} | 10^{-7} | 10^{-8} | 10^{-9}

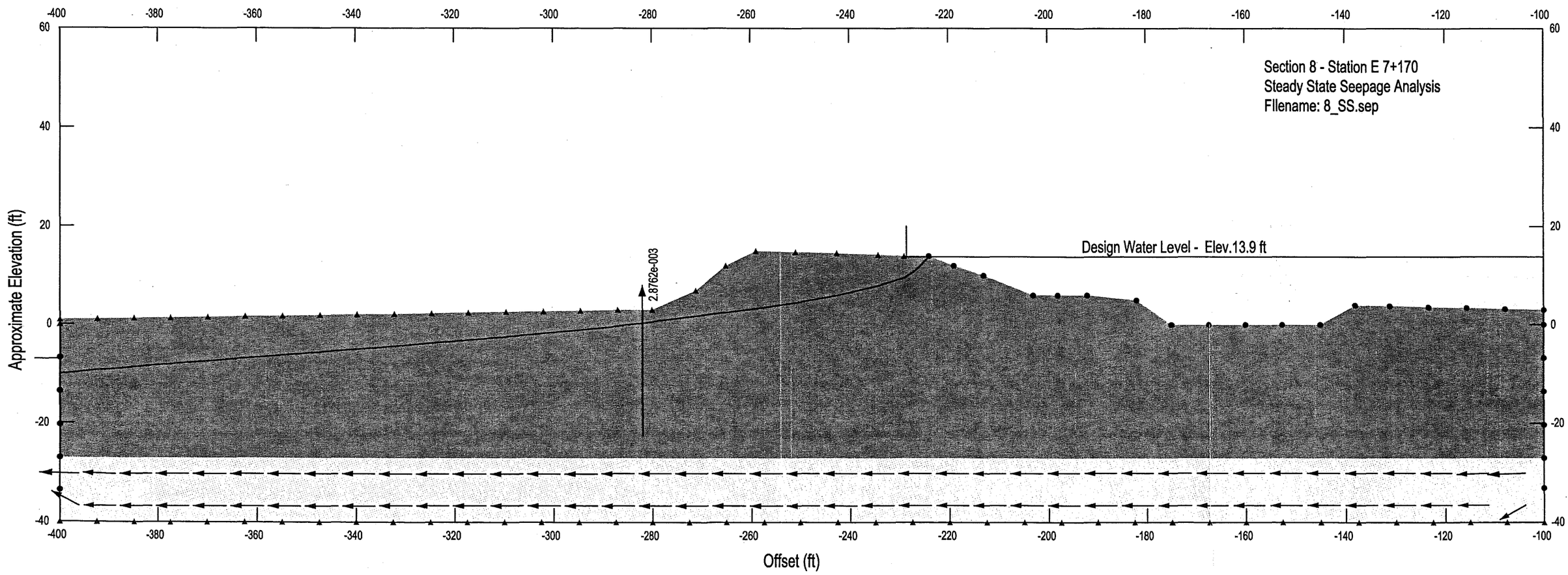
Drainage	Good		Poor	Practically Impervious
Soil types	Clean gravel	Clean sands, clean sand and gravel mixtures	Very fine sands, organic and inorganic silts, mixtures of sand silt and clay, glacial till, stratified clay deposits, etc.	"Impervious" soils, e.g., homogeneous clays below zone of weathering
			"Impervious" soils modified by effects of vegetation and weathering	
Direct determination of k	Direct testing of soil in its original position—pumping tests. Reliable if properly conducted. Considerable experience required			
	Constant-head permeameter. Little experience required			
Indirect determination of k		Falling-head permeameter. Reliable. Little experience required	Falling-head permeameter. Unreliable. Much experience required	Falling-head permeameter. Fairly reliable. Considerable experience necessary
	Computation from grain-size distribution. Applicable only to clean cohesionless sands and gravels			Computation based on results of consolidation tests. Reliable. Considerable experience required

After A. Casagrande and R. E. Fadum

FIG. 2.8 Permeability and drainage characteristics of soils. (Table 6, p. 48, of *Soil Mechanics in Engineering Practice*, by K. Terzaghi and R. B. Peck, John Wiley and Sons, New York, 1948.)

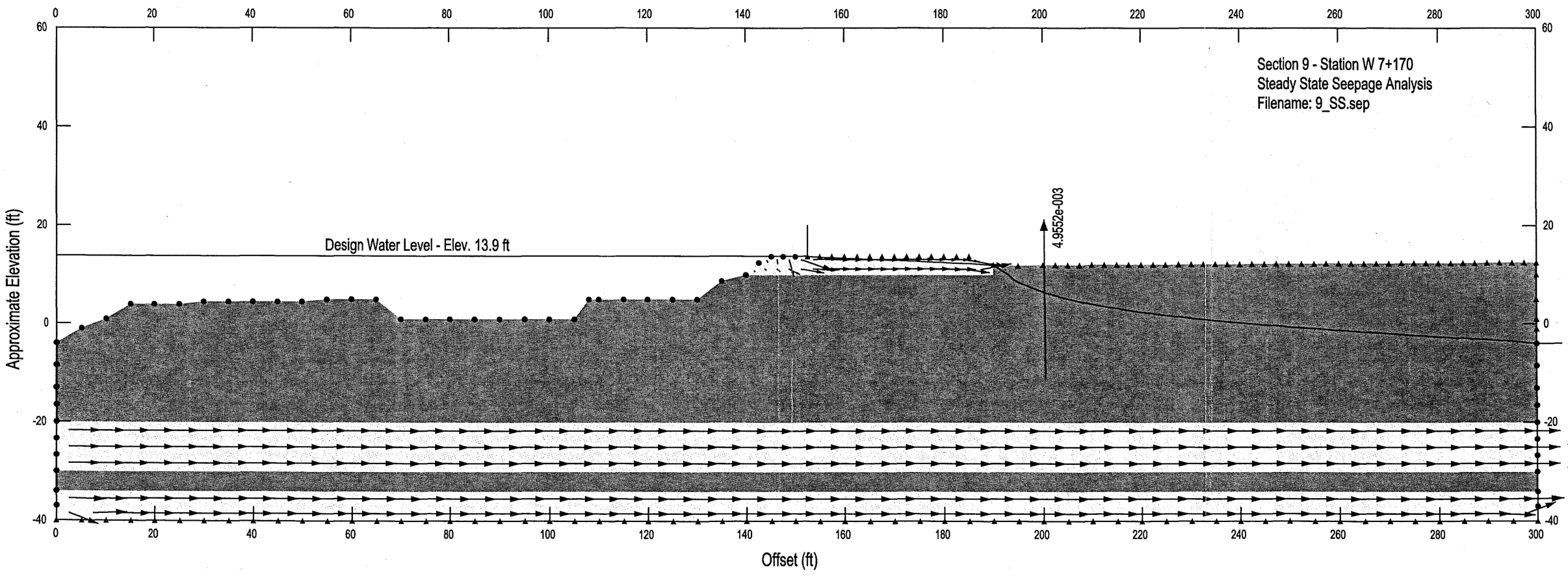
Q:\CADD2\01FILES\95-00112030.00\244\FIG38.DWG

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	PERMEABILITY AND DRAINAGE CHARACTERISTICS OF SOILS	Figure 38
6/1/02			
URS			



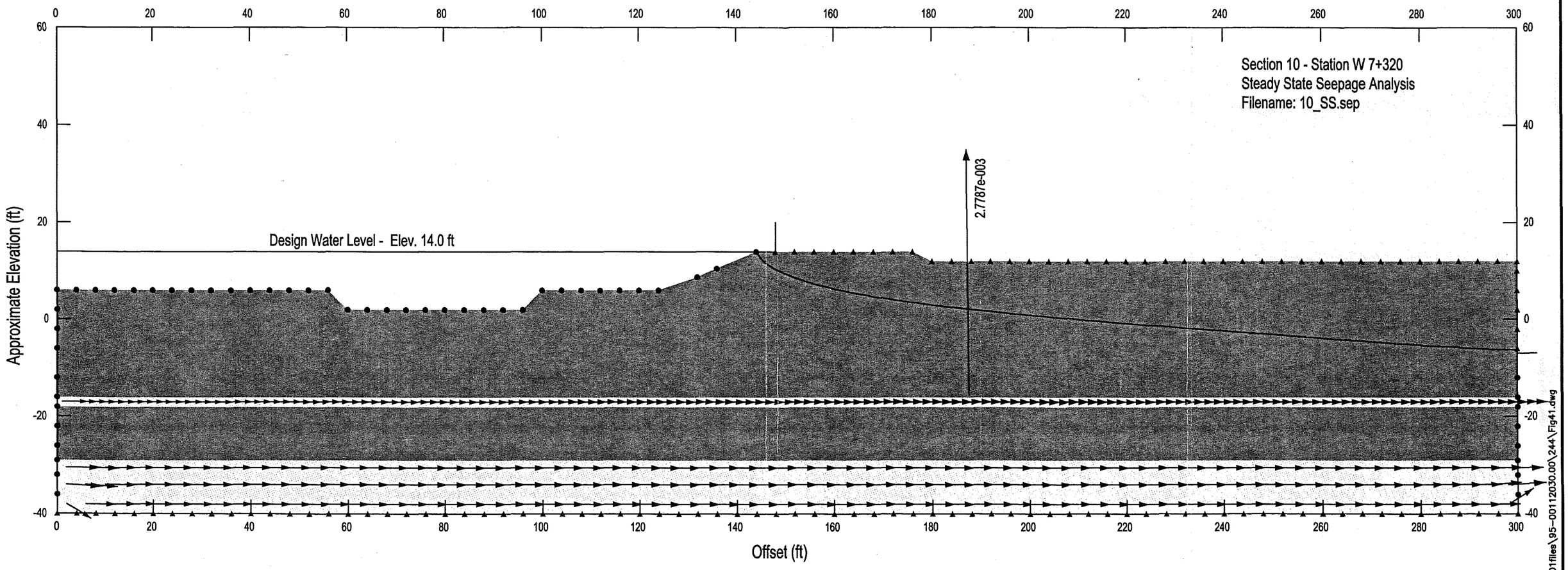
q:\cadd2\01files\95-00112030.00\244\Fig39.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STEADY STATE SEEPAGE ANALYSIS CROSS SECTION 8	Figure 39
6/1/02	URS		



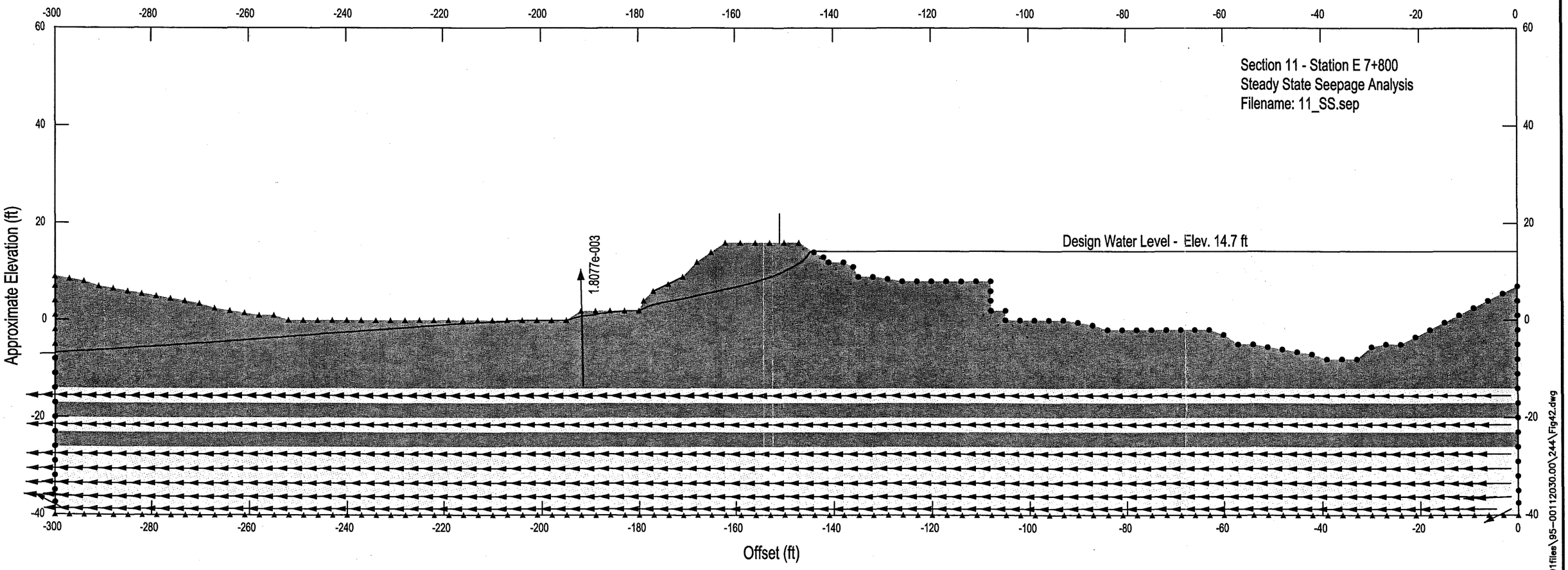
q:\cadd2\01files\95-00112030.00\244\Fig40.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STEADY STATE SEEPAGE ANALYSIS CROSS SECTION 9	Figure 40
6/1/02	URS		



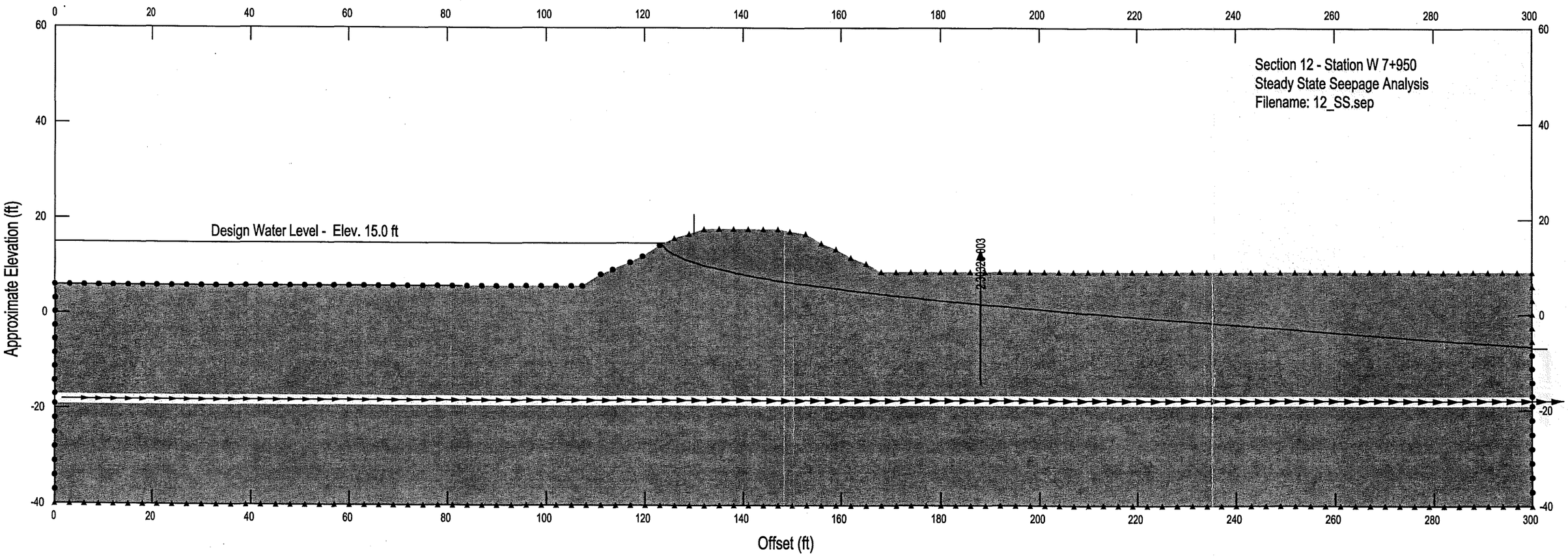
q:\cadd2\01files\95-00112030.00\244\Fig41.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STEADY STATE SEEPAGE ANALYSIS CROSS SECTION 10	Figure 41
6/1/02	URS		



q:\cadd2\01files\95-00112030.00\244\Fig42.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STEADY STATE SEEPAGE ANALYSIS CROSS SECTION 11	Figure 42
6/1/02	URS		

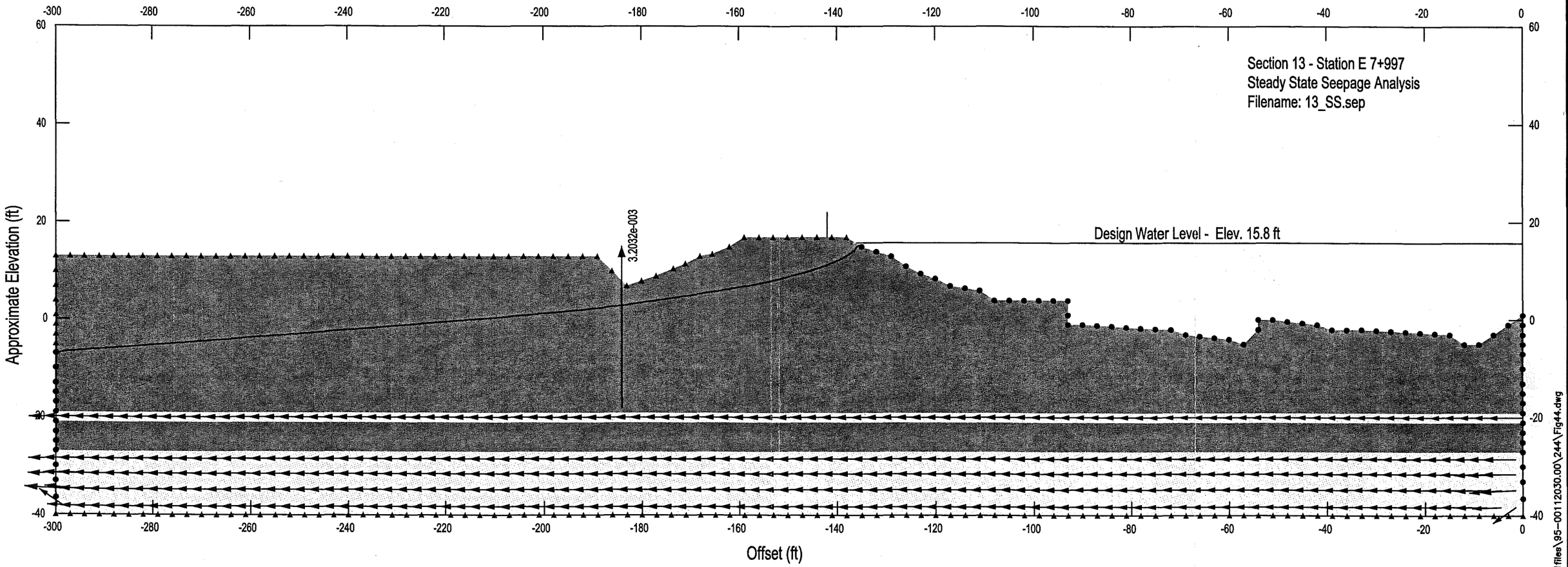


q:\cadd2\01files\95-00112030.00\244\Fig43.dwg

LEGEND:

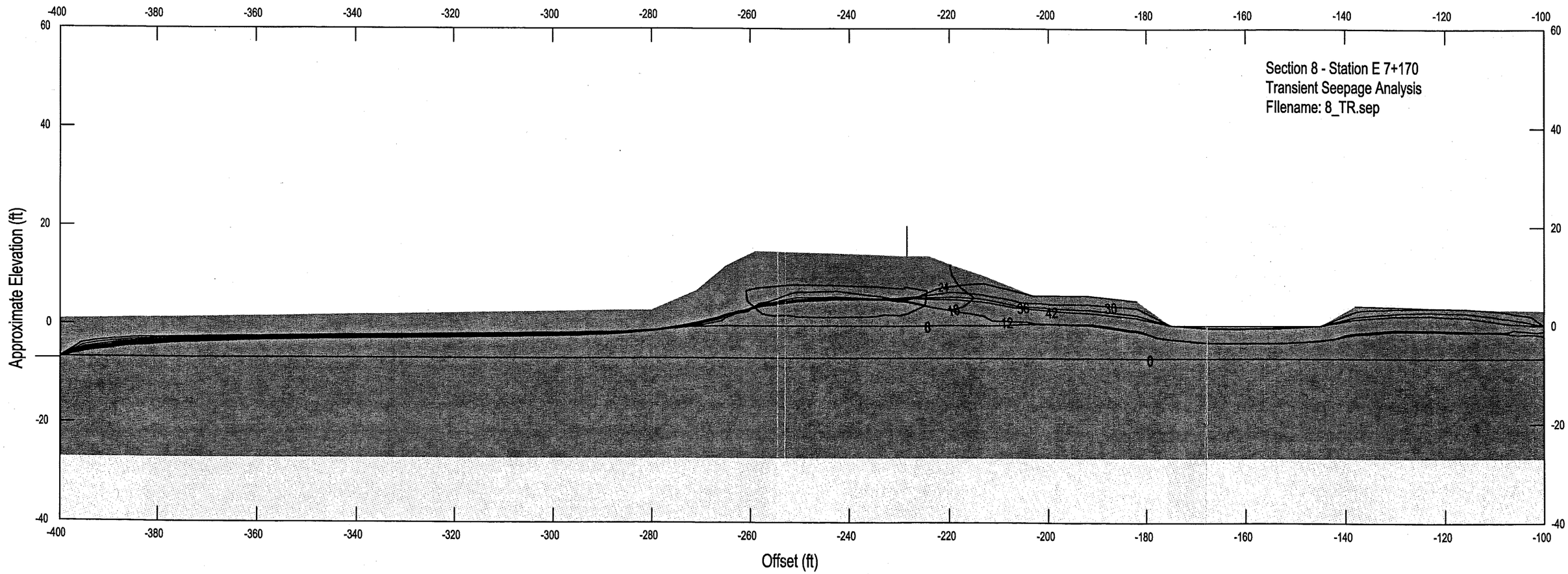
24 PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STEADY STATE SEEPAGE ANALYSIS CROSS SECTION 12	Figure 43
6/1/02	URS		



q:\cadd2\01files\95-00112030.00\244\Fig44.dwg

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	STEADY STATE SEEPAGE ANALYSIS CROSS SECTION 13	Figure 44
6/1/02	URS		

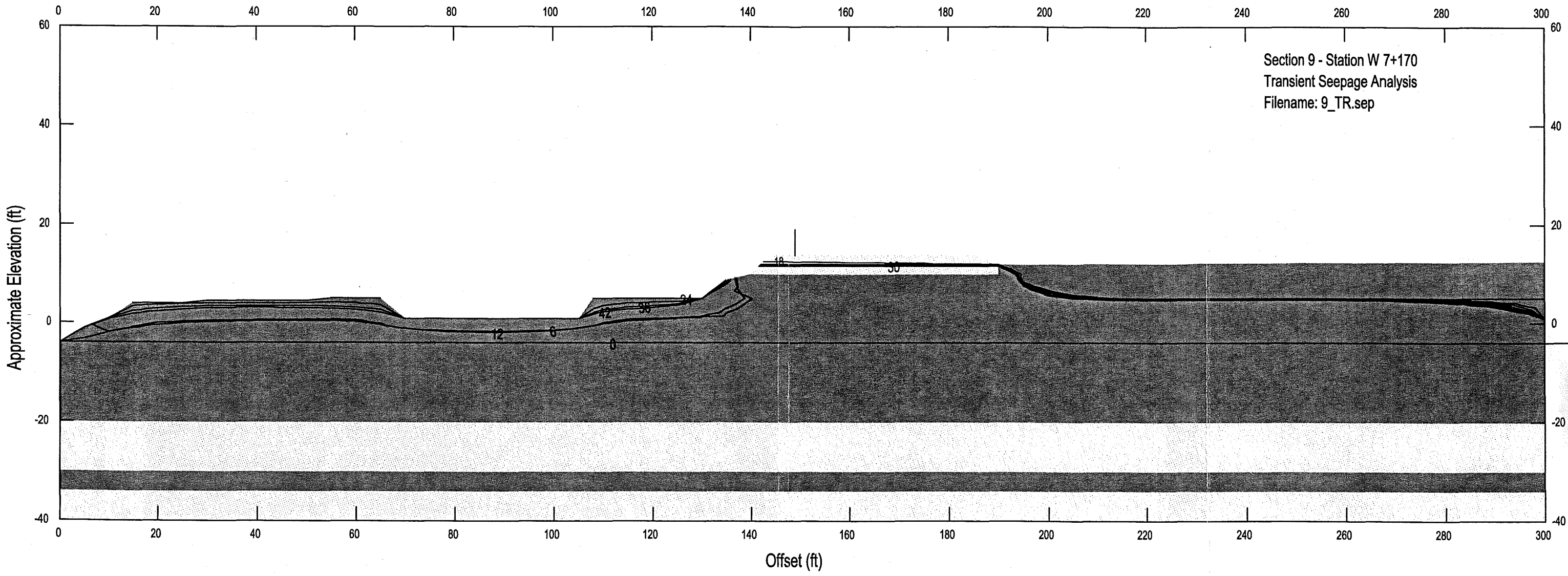


q:\cadd2\01filee\95-00112030.00\244\Fig45.dwg

LEGEND:

24 PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 8	Figure 45
6/1/02	URS		

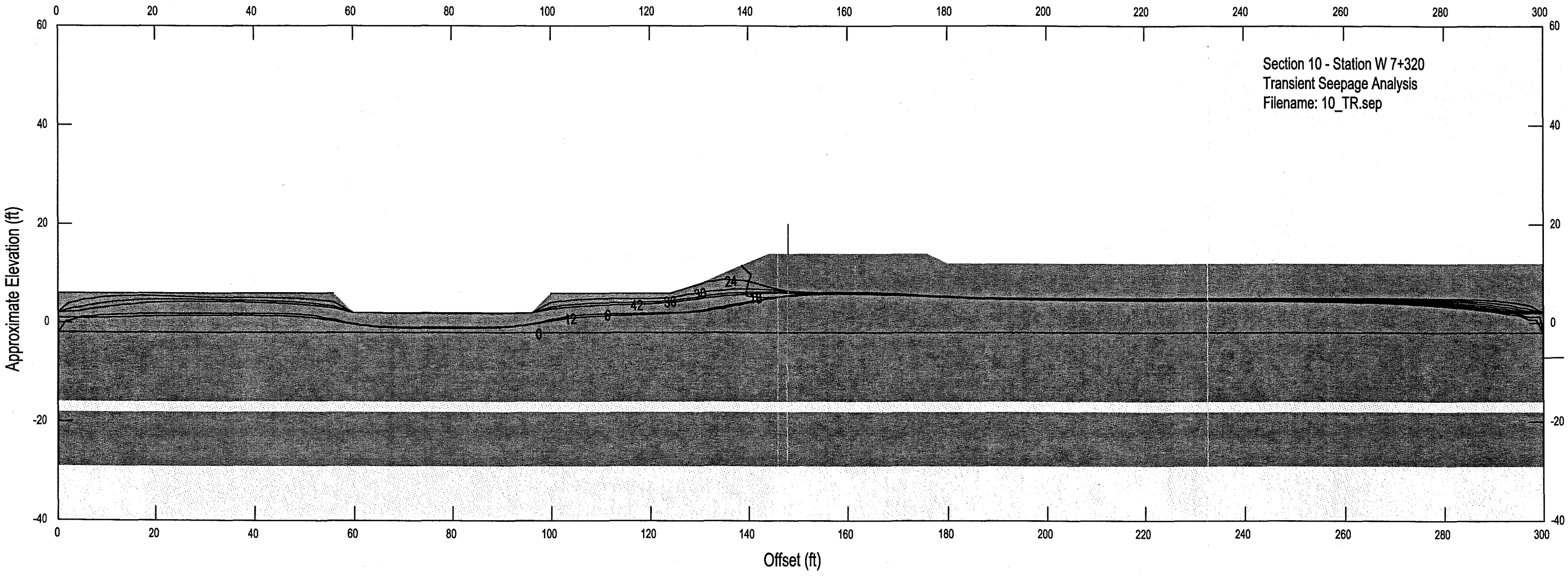


q:\cadd2\01files\95-00112030.00\244\Fig46.dwg

LEGEND:

24 PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 9	Figure 46
6/1/02	URS		



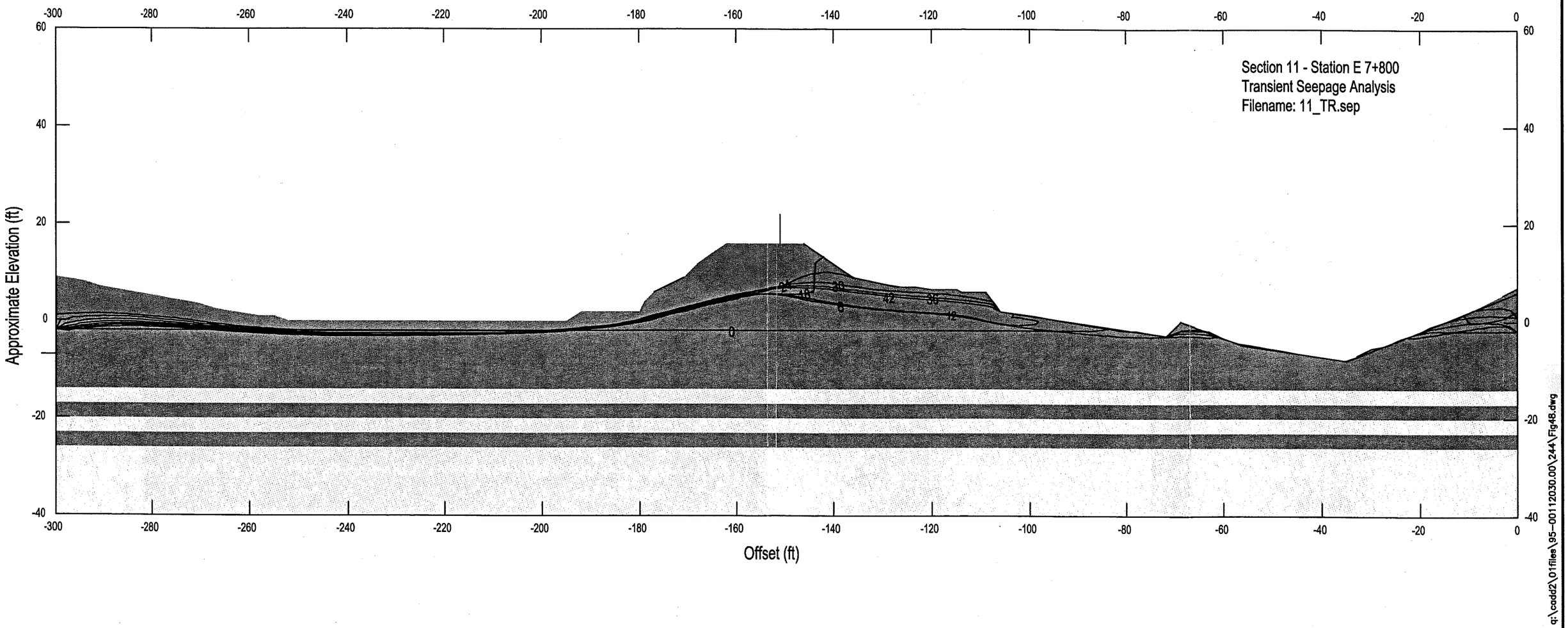
LEGEND:

24 PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 10
6/1/02	URS	

Figure 47

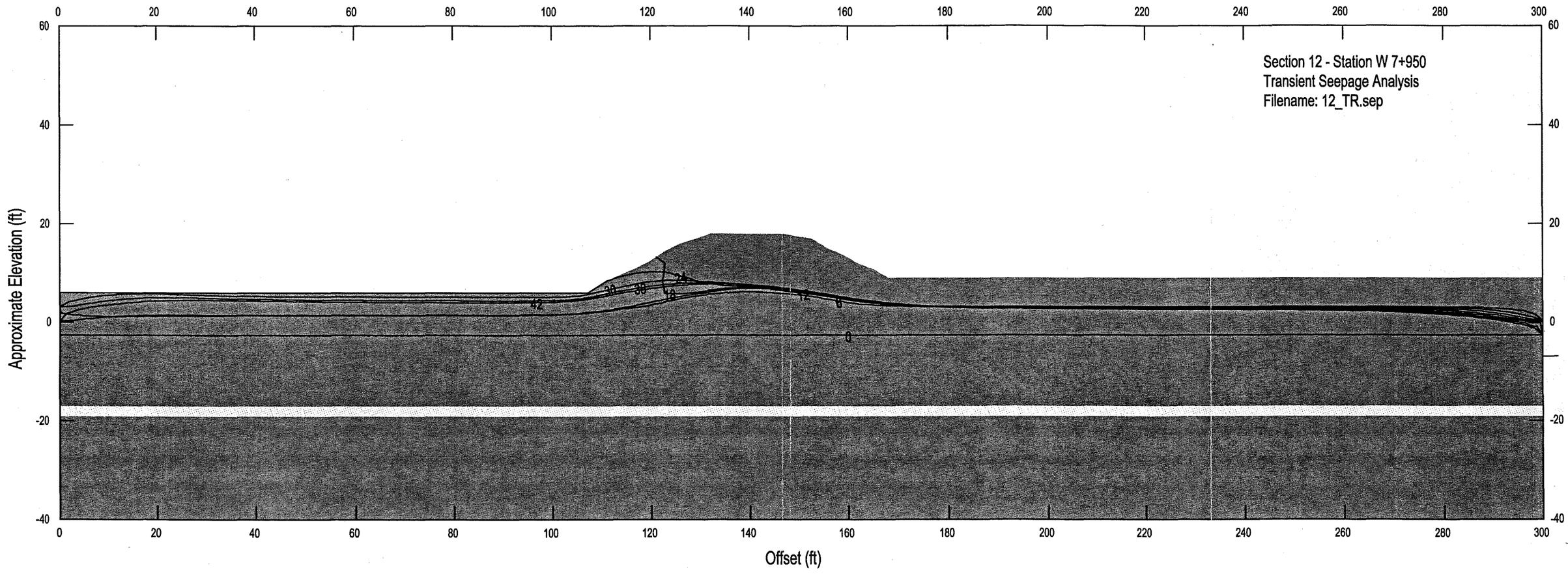
q:\cadd2\01files\95-00112030.00\244\Fig47.dwg



LEGEND:

24 PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 11	Figure 48
6/1/02	URS		

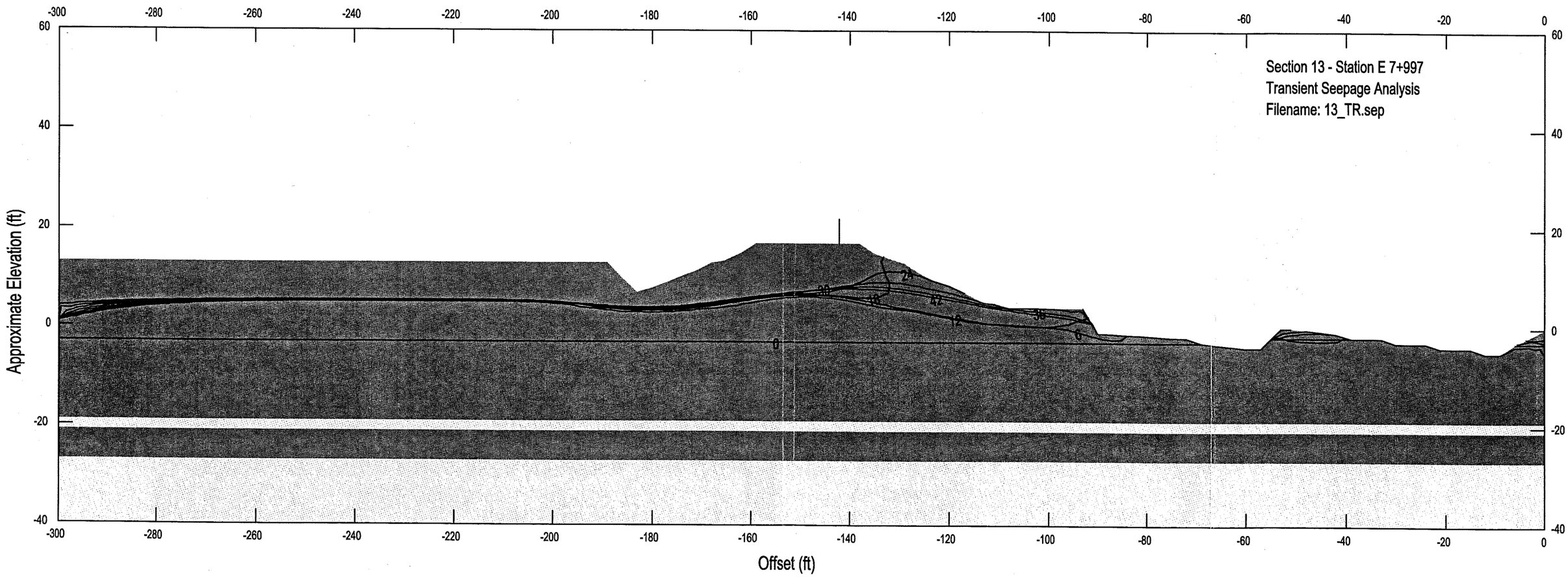


q:\cadd2\01files\95-00112030.00\244\Fig49.dwg

LEGEND:

24 PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 12	Figure 49
6/1/02	URS		

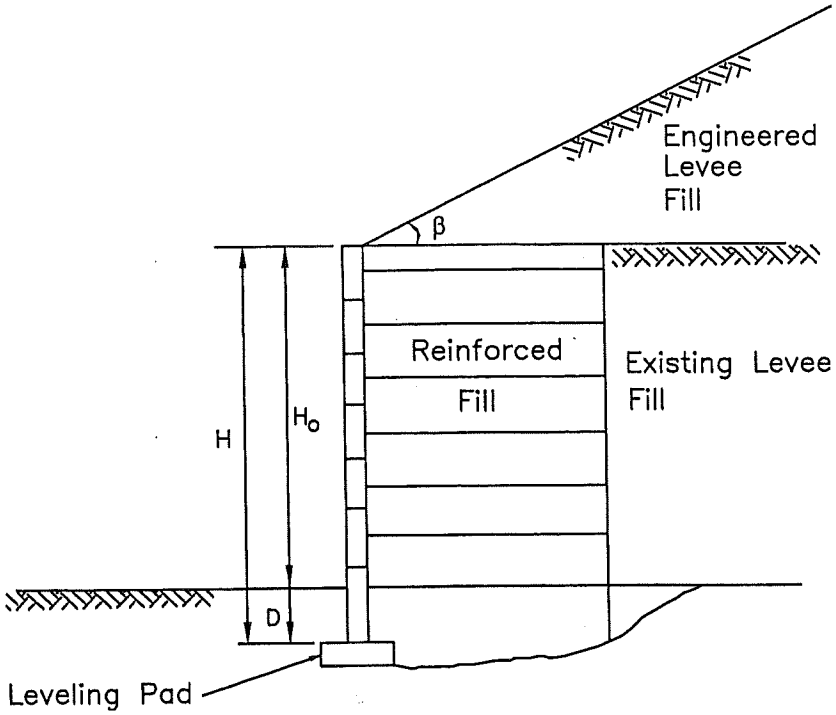


q:\cadd2\01files\95-00112030.00\244\Fig50.dwg

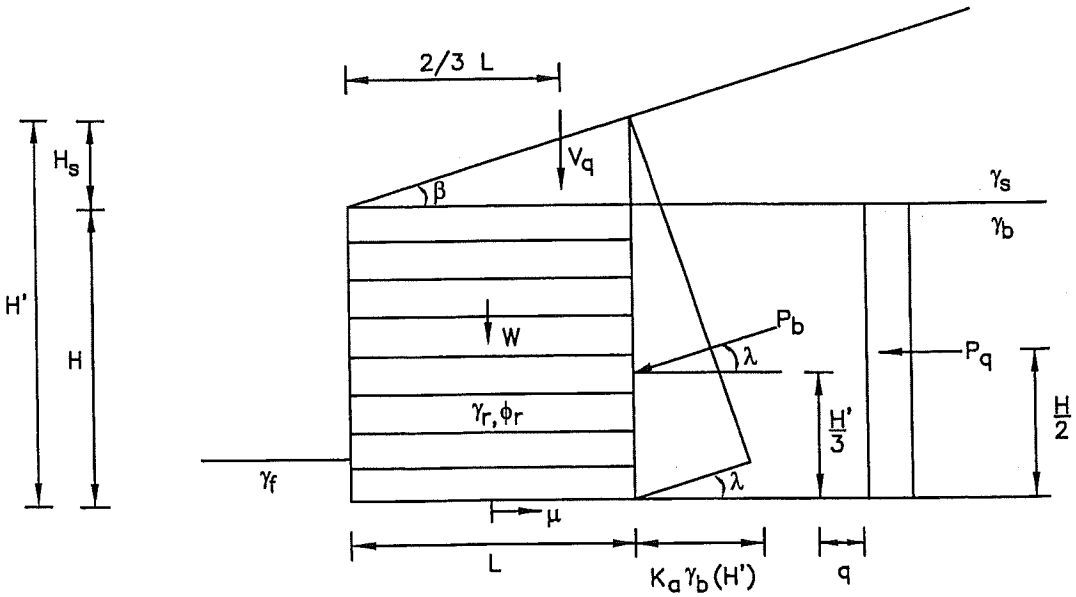
LEGEND:

24 ⌒ PHREATIC SURFACE AT TIME STEP 24 (48 HOURS)

Project No. 95-00112030.00	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara County, California	TIME DEPENDENT PHREATIC SURFACE LINES, CROSS SECTION 13	Figure 50
6/1/02	URS		



MSE WALL CONFIGURATION



$P_b = 0.5 K_a \gamma_b (H')^2$

$P_q = q H$

$\mu = 0.48$

$W = \gamma_r H L$

$V_q = 0.5 \gamma_s H_s L$

$H_s = L \tan \beta$

Assumptions:
(1) $\gamma_b = \gamma_s$
(2) $q = 0.6 \gamma_s$ (kN/m³)
(3) $\phi_r > 34°$
(4) $\beta = \lambda$ (for extensible reinforcement)

EXTERNAL LOADING DIAGRAM

Q:\CADD2\01FILES\95-00112030.00\244\msewall51.dwg

Project No. 95-00112030.00 6/1/02	LOWER GUADALUPE RIVER FLOOD CONTROL PROJECT Santa Clara, California	TYPICAL MSE WALL CONFIGURATION WITH FORCE DIAGRAM	Figure 51
URS			

TEMP002

Appendix A

APPENDIX A

1997 Guadalupe Mitigation Levee Drawings and 1997 Geotechnical Investigation Report

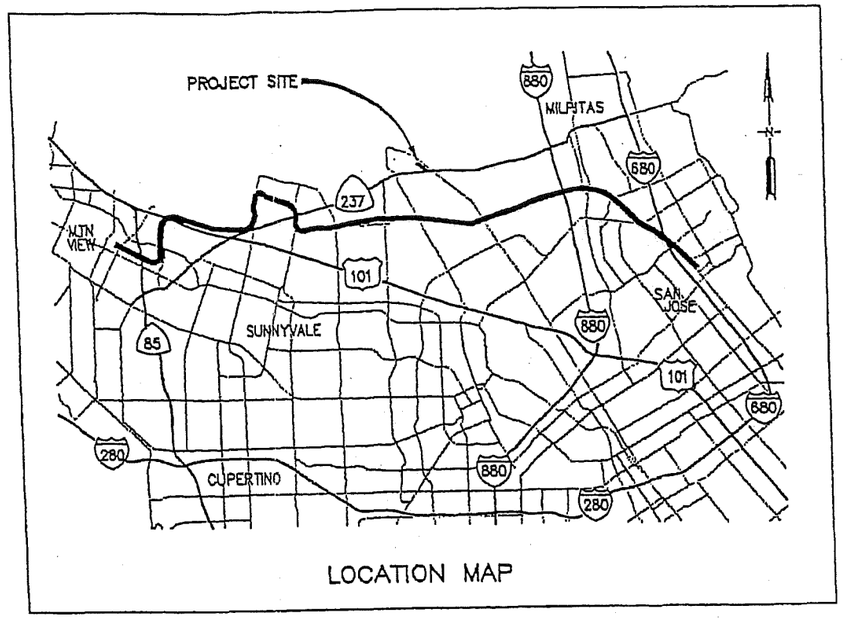
SANTA CLARA VALLEY
TRANSPORTATION AUTHORITY

TASMAN CORRIDOR PROJECT

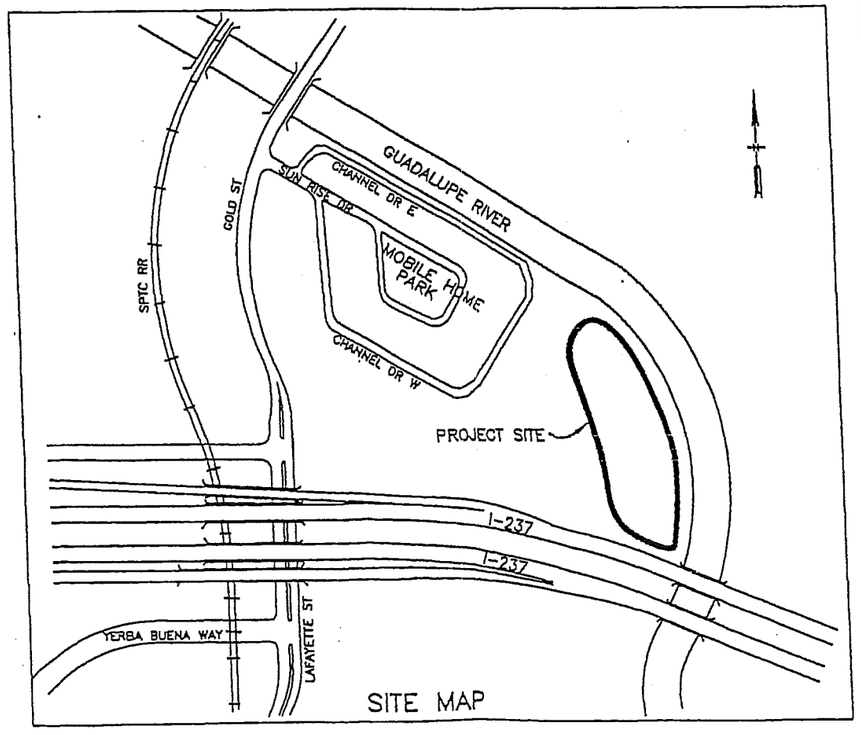
GUADALUPE MITIGATION SITE
CONTRACT

DRAWING INDEX

SHT. NO.	DWG NO.	TITLE
CIVIL		
1	G-1	COVER SHEET
2	C-1	LAYOUT PLAN
3	C-2	PROFILE AND TYPICAL SECTIONS
4	C-3	CROSS SECTIONS
5	C-4	CROSS SECTIONS
6	C-5	CROSS SECTIONS
7	C-6	CROSS SECTIONS
8	C-7	CULVERT PLAN, SECTIONS AND DETAILS
9	C-8	SECTIONS AND DETAILS



LOCATION MAP



SITE MAP

RECEIVED
SEP 24 2001
BY: Old Mathy

PRELIMINARY

NO.	REVISIONS	DATE

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY

DESIGNED	FRB	CADD FILE NAME	5413G01A
DRAWN	FRB	CADD FILE DATE	1/16/97
CHECKED	GWB	PLOT DATE	1/16/97
SCALE	AS SHOWN	PCN NO.	

SUBMITTED: AN WEST INC. Consulting Engineers, 4125 Lakeside Drive, Richmond, CA 94805-1942

APPROVED

TASMAN CORRIDOR PROJECT
GENERAL

TITLE SHEET

SHEET XX OF XX
DRAWING NO. G-1
REVISION X

BOARD APPROVAL DATE | CONTRACT NO. | FILE NO.

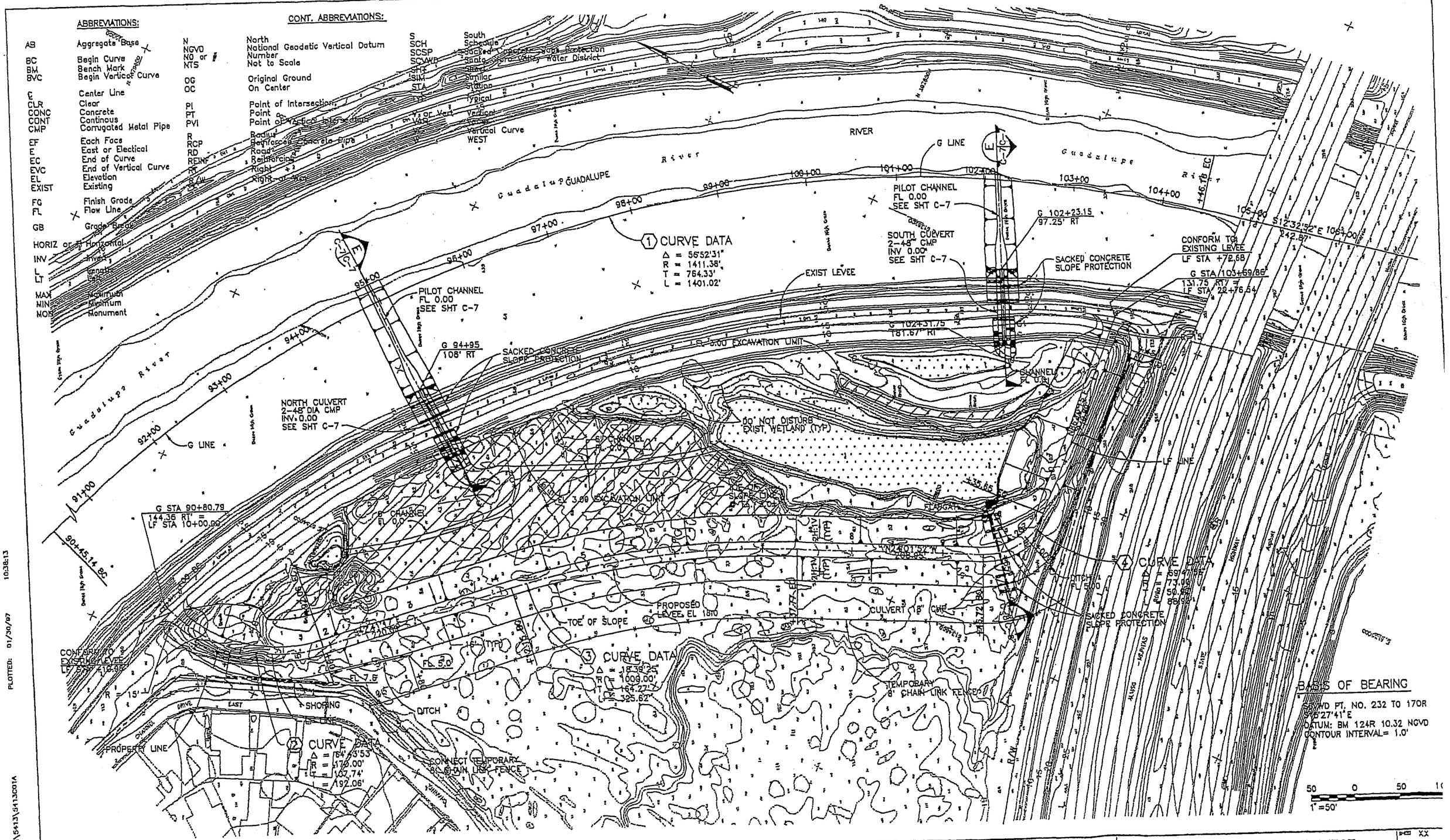
FILE NAM. .\PROJECTS\5413\5413G01A PLOTTED: 01/30/97 13:28:51

ABBREVIATIONS:

AB	Aggregate Base
BC	Begin Curve
BM	Bench Mark
BVC	Begin Vertical Curve
℄	Center Line
CLR	Clear
CONC	Concrete
CONT	Continous
CMP	Corrugated Metal Pipe
EF	Each Face
E	East or Electical
EC	End of Curve
EVC	End of Vertical Curve
EL	Elevation
EXIST	Existing
FG	Finish Grade
FL	Flow Line
GB	Grade Break
HORIZ or H	Horizontal
INV	Invert
L	Length
LT	Left
MAX	Maximum
MIN	Minimum
MON	Monument

CONT. ABBREVIATIONS:

N	North
NGVD	National Geodetic Vertical Datum
NO or #	Number
NTS	Not to Scale
OG	Original Ground
OC	On Center
PI	Point of Intersection
PT	Point
PVI	Point of Vertical Intersection
R	Radius
RCP	Reinforced Concrete Pipe
RD	Road
REINF	Reinforcing
RT	Right
R/W	Right of Way
S	South
SCH	Schedule
SCSP	Sacked Concrete Slope Protection
SCVWD	Santa Clara Valley Water District
SHT	Sheet
SIM	Similar
STA	Station
TYP	Typical
V or Vert	Vertical
VAR	Varies
VC	Vertical Curve
W	WEST



DESIGNED	AAD
DRAWN	FRB
CHECKED	GWB
SCALE	1"=50'
CADD FILE NAME	5413C01A
CADD FILE DATE	12/30/96
PLOT DATE	XXXXXX
FCA NO.	XXYY

SUBMITTED: ANWEST INC. Consulting Engineers, 4123 Lakeside Drive, Richmond, CA 94806-1942

APPROVED

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY

TASMAN CORRIDOR PROJECT

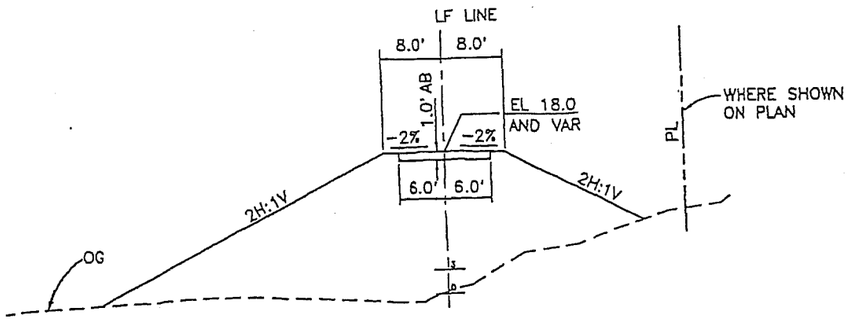
GUADALUPE MITIGATION SITE

LAYOUT PLAN

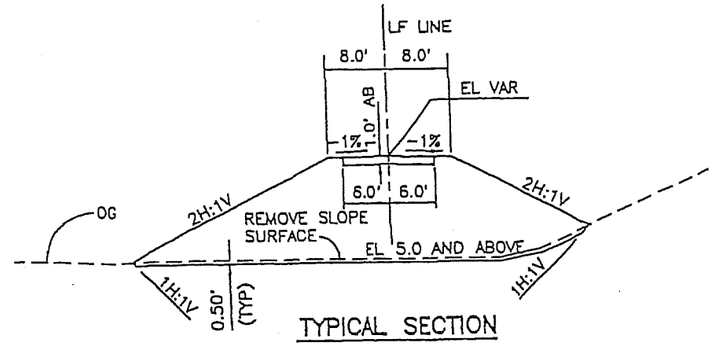
PRELIMINARY

SHEET	XX
OF	XX
DRAWING NO.	C-1
REVISION	
NCVD APPROVAL DATE	XXXX
CONTRACT NO.	XXXXXX
FILE NO.	XXXXXX

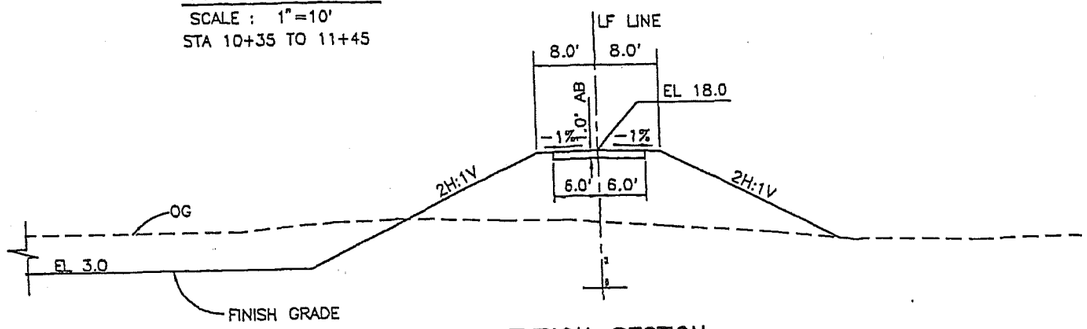
FILE NAME: C:\PROJECTS\5413\5413C01A PLOTTED: 01/30/97 10:38:13



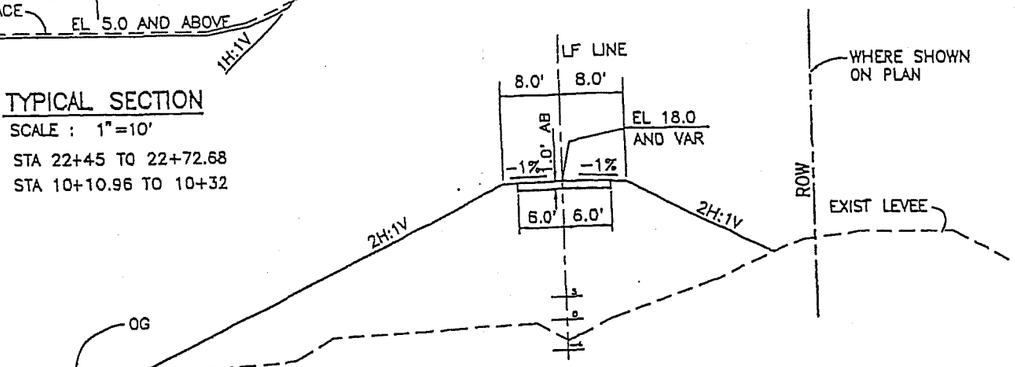
TYPICAL SECTION
SCALE : 1"=10'
STA 10+35 TO 11+45



TYPICAL SECTION
SCALE : 1"=10'
STA 22+45 TO 22+72.68
STA 10+10.96 TO 10+32

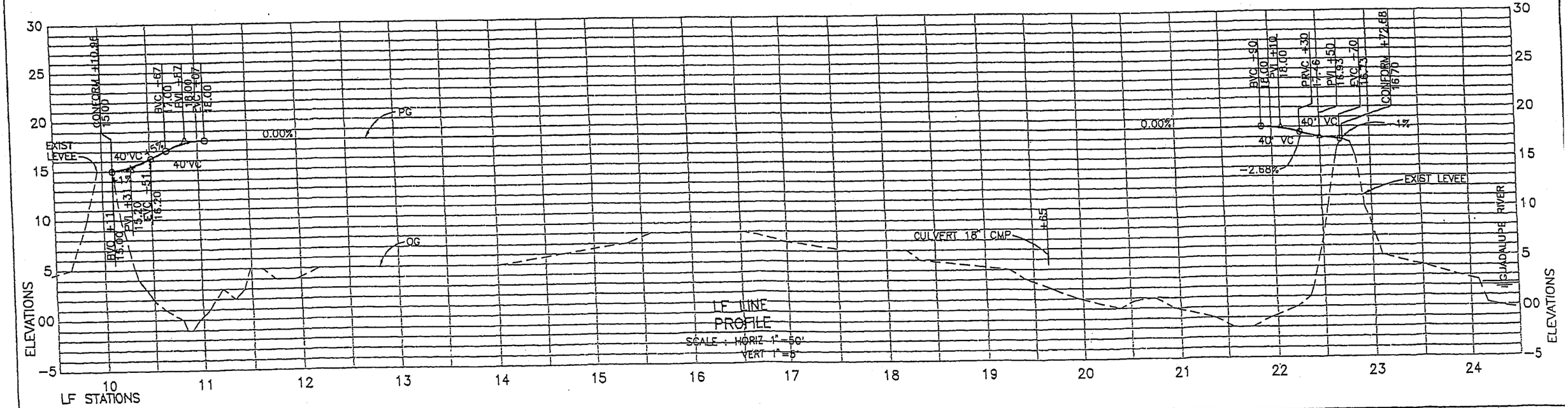


TYPICAL SECTION
SCALE : 1"=10'
STA 11+45 TO 19+30



TYPICAL SECTION
SCALE : 1"=10'
STA 19+30 TO 22+45

10 0 10 20
1"=10'
50 0 50 100
1"=50'



NO.	REVISIONS	DATE

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY

DESIGNED FRB	CADD FILE NAME 5413C02A	SUBMITTED	APPROVED
DRAWN FRB	CADD FILE DATE 12/31/96		
CHECKED GWB	PLOT DATE		
SCALE AS SHOWN	PCA NO. XXXX		

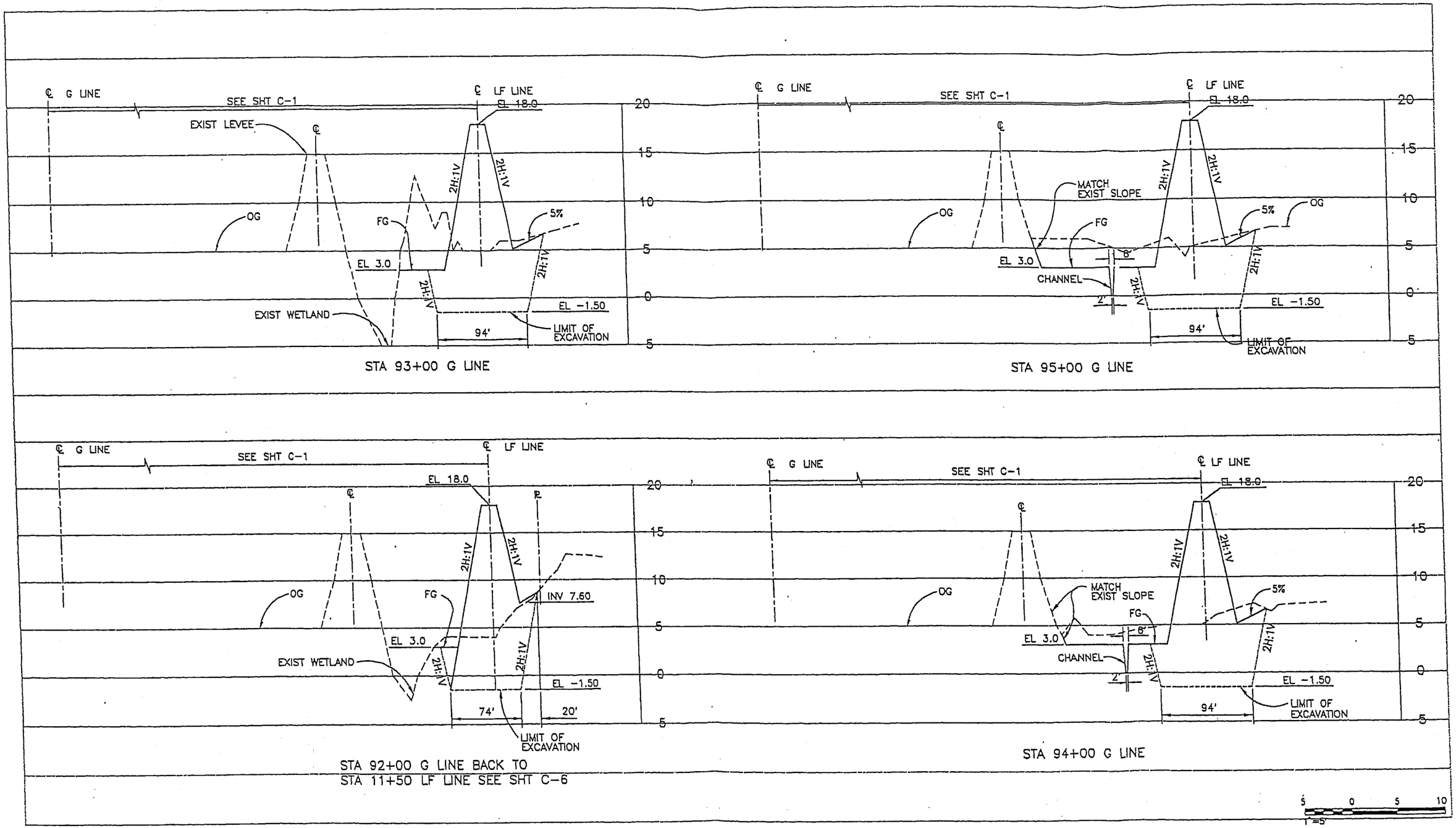
AMWEST INC.
Consulting Engineers
4123 Lakeside Drive
Richmond, CA 94806-1942

TASMAN CORRIDOR PROJECT
GUADALUPE MITIGATION SITE
PROFILE AND TYPICAL SECTIONS
PRELIMINARY

SHEET XX OF XX
DRAWING NO. C-2
REVISION

BOARD APPROVAL DATE XXX | CONTRACT NO. XXXXXX | FILE NO. XXXXXX

FILE NAME: C:\PROJE\13\5413C02A PLOTTED: 01/30/97 11:53:24



NO.	REVISIONS	DATE

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY

DESIGNED RAB	CADD FILE NAME 5413C03A	SUBMITTED	APPROVED
DRAWN DDE	CADD FILE DATE 12/31/96		
CHECKED GWB	PLOT DATE XXXXXX		
SCALE AS SHOWN	PCA NO. XXYY		

AN WEST INC.
Consulting Engineers
4123 Lakeside Drive
Richmond, CA 94806-1942

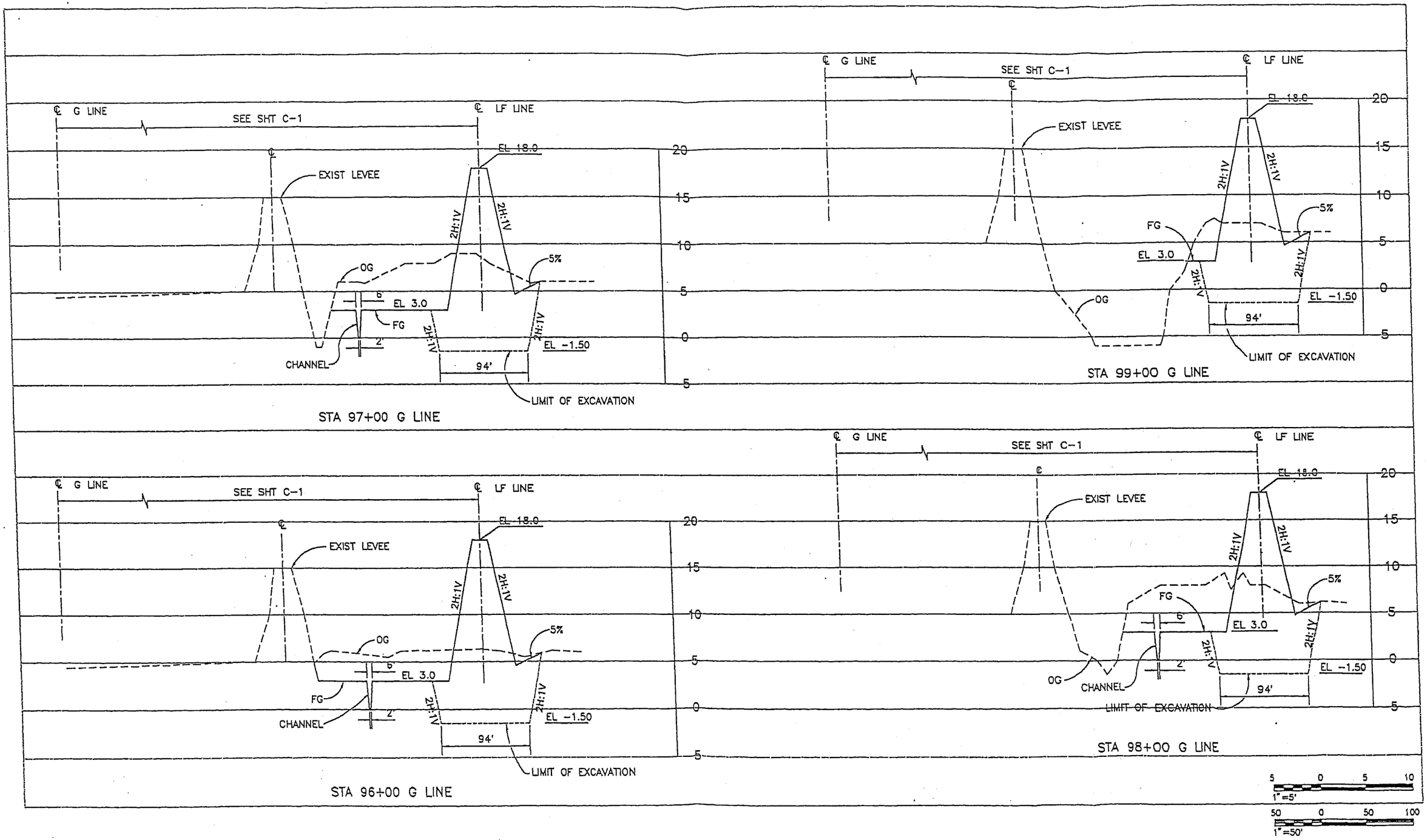
TASMAN CORRIDOR PROJECT
GUADALUPE MITIGATION SITE
CROSS SECTIONS

PRELIMINARY

BOARD APPROVAL DATE XXXX	CONTRACT NO. XXXXXX	FILE NO. XXXXXX

SHEET XX OF XX
DRAWING NO. C-3
REVISION

FILE NAME: IECTS\5413\5413C03A PLOTTED: 01/30/97 11:52:34



NO.	REVISIONS	DATE

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY

DESIGNED RAB	CADD FILE NAME 5413C04A	SUBMITTED	APPROVED
DRAWN FRB	CADD FILE DATE 12/31/96		
CHECKED GWB	PLOT DATE XXXXXX		
SCALE AS SHOWN	PGA NO. XXYY		

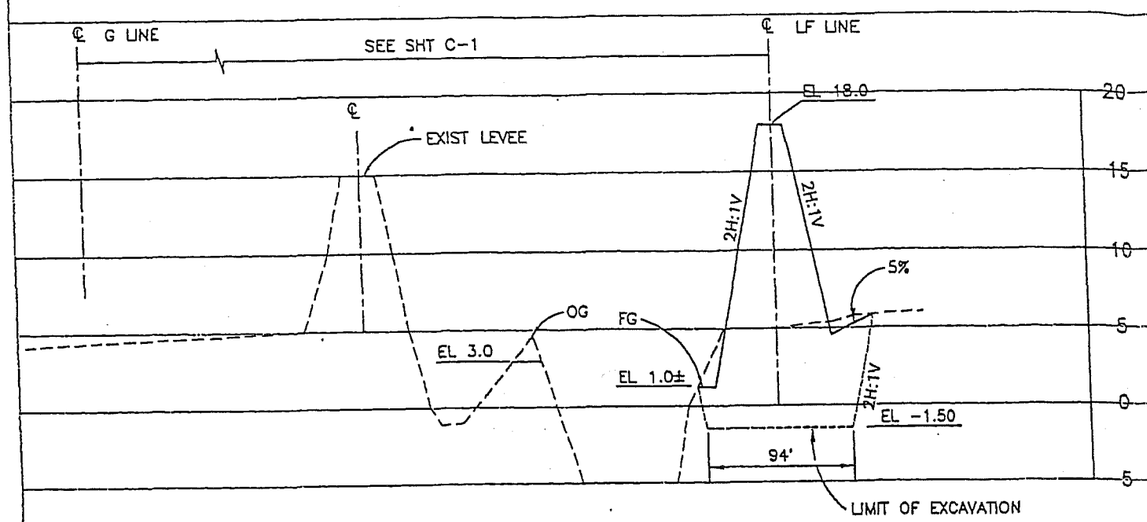
AN WEST INC.
Consulting Engineers
4123 Lakeside Drive
Richmond, CA 94806-1942

TASMAN CORRIDOR PROJECT
GUADALUPE MITIGATION SITE
CROSS SECTIONS
PRELIMINARY

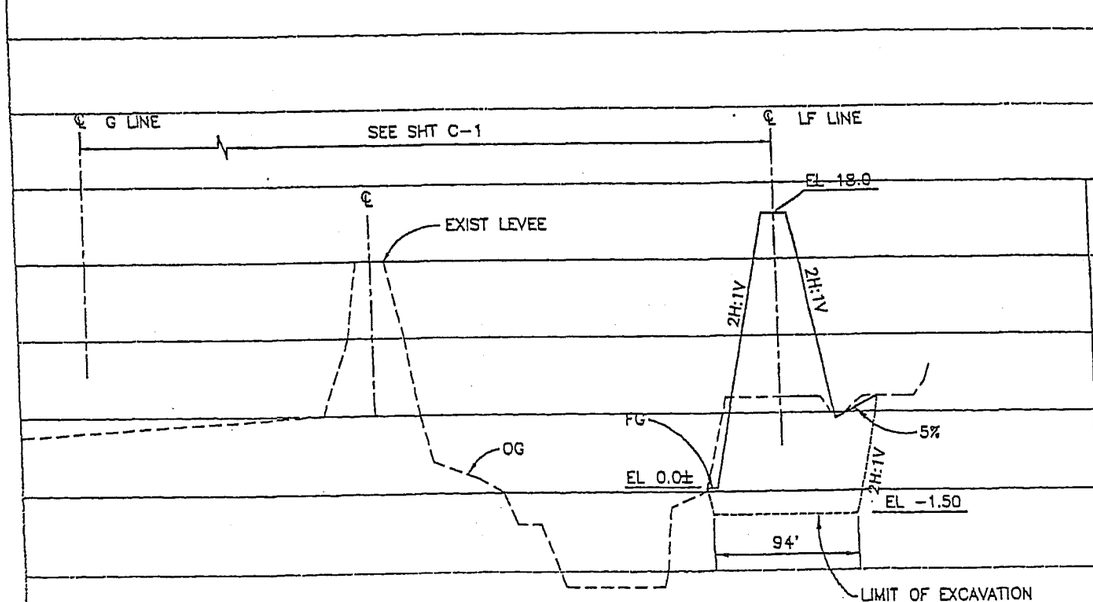
BOARD APPROVAL DATE XXX	CONTRACT NO. XXXXXX	FILE NO. XXXXXX

SHEET XX OF XX
DRAWING NO. C-4
REVISION

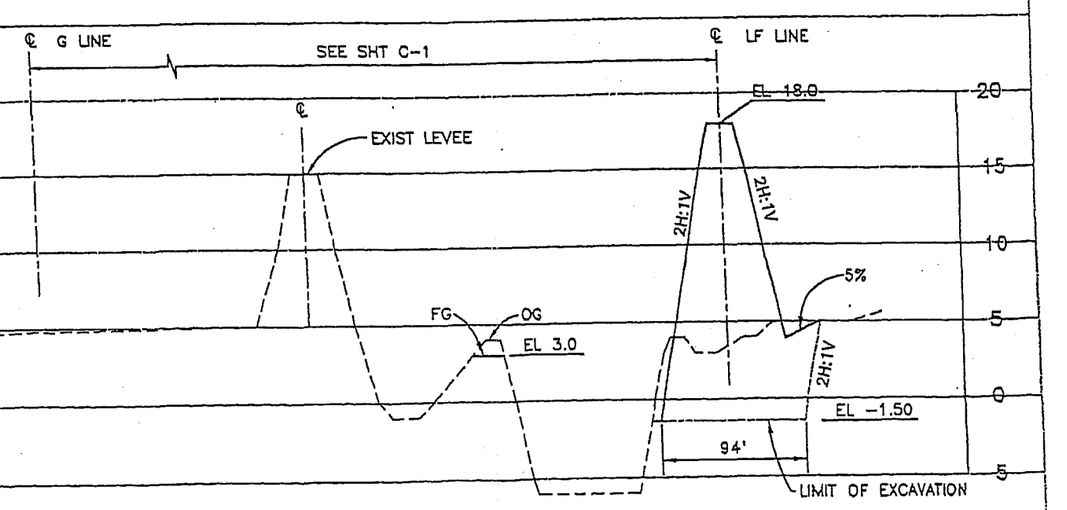
FILE NAME: C:\PRG. A\5413\5413C04A
PLOTTED: 01/30/97 11:51:47



STA 101+00 G LINE



STA 100+00 G LINE



STA 102+00 G LINE NEXT TO
~~STA 20+00 LF LINE SEEE SHT C-6~~

5 0 5 10
1" = 5'

50 0 50 100
1" = 50'

NO.	REVISIONS	DATE

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY

DESIGNED RAB	CADD FILE NAME 5413C05A	SUBMITTED	APPROVED
DRAWN FRB	CADD FILE DATE 12/31/96		
CHECKED GWB	PLOT DATE XXXXXX		
SCALE AS SHOWN	PCA NO. XXYY		

AN WEST INC.
Consulting Engineers
4123 Lakeside Drive
Richmond, CA 94806-1942

TASMAN CORRIDOR PROJECT
GUADALUPE MITIGATION SITE

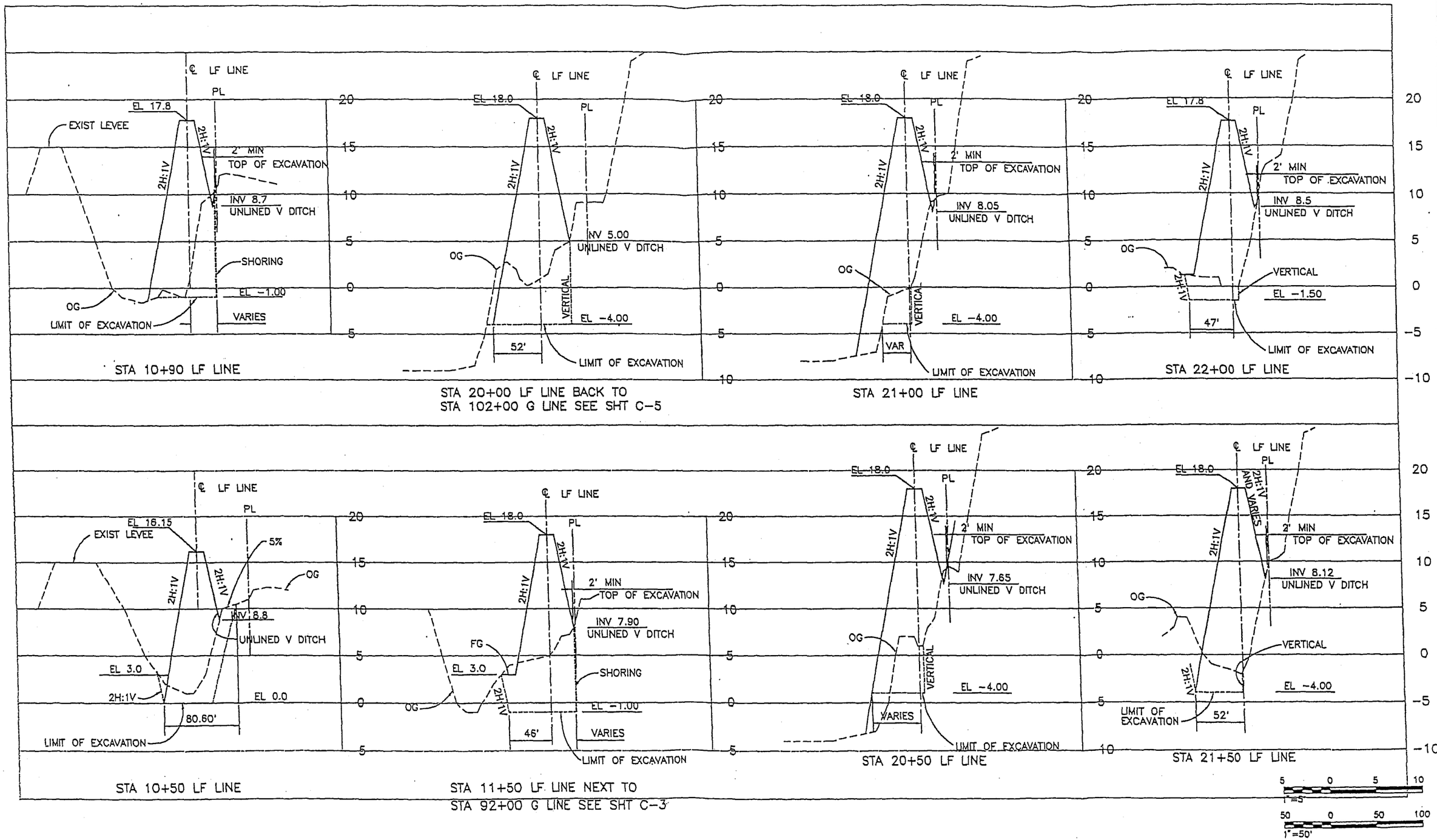
CROSS SECTIONS

PRELIMINARY

BOARD APPROVAL DATE XXX	CONTRACT NO. XXXXXX	FILE NO. XXXXXX

SHEET XX OF XX
DRAWING NO. C-5
REVISION

FILE NAME: G:\PROJECTS\5413\5413C05A
PLOTTED: 01/30/97 11:50:48



NO.	REVISIONS	DATE

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY

DESIGNED FRB	CADD FILE NAME 5413C06A	SUBMITTED	APPROVED
DRAWN FRB	CADD FILE DATE 12/30/96		
CHECKED GWB	PLOT DATE XXXXXX		
SCALE AS SHOWN	FCA NO. XXYY		

AMWEST INC.
Consulting Engineers
4123 Lakeside Drive
Richmond, CA 94806-1942

TASMAN CORRIDOR PROJECT
GUADALUPE MITIGATION SITE

CROSS SECTIONS

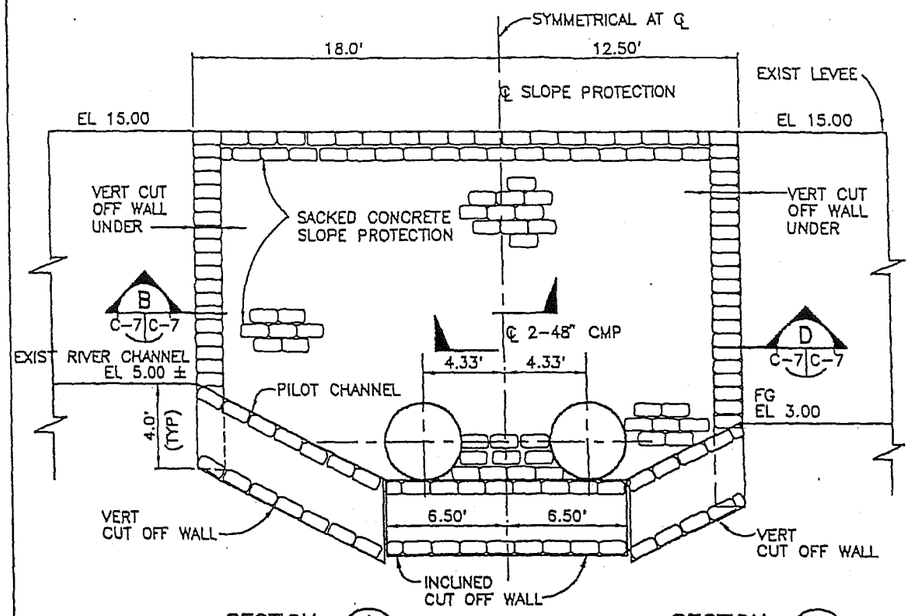
PRELIMINARY

BOARD APPROVAL DATE XXX	CONTRACT NO. XXXXXX	FILE NO. XXXXXX

SHEET XX OF XX
DRAWING NO. C-6
REVISION

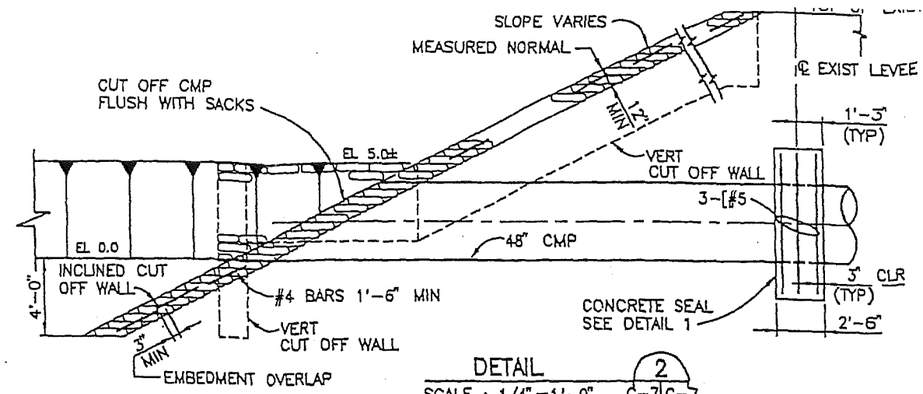
PLOTTED: 01/30/97 11:50:04

FILE NAME: c:\PROJECTS\5413\5413C06A

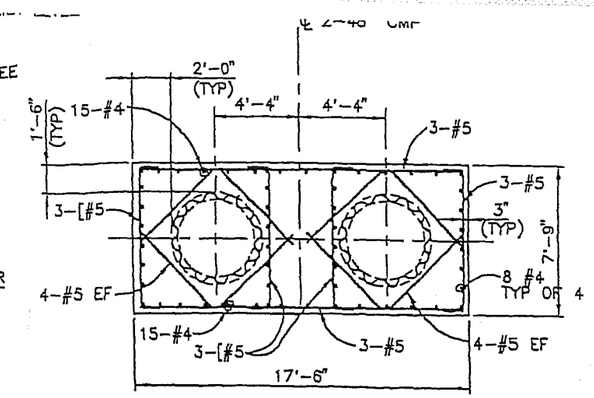


SECTION A
SCALE : 1"=4'
C-7|C-7

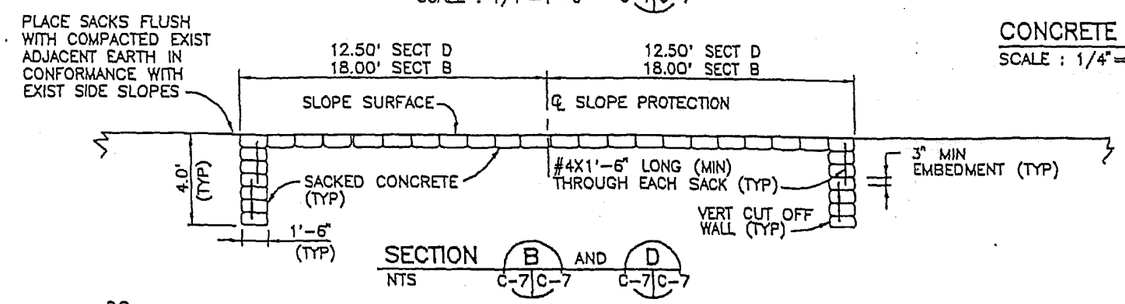
SECTION C
SCALE : 1"=4'
C-7|C-7



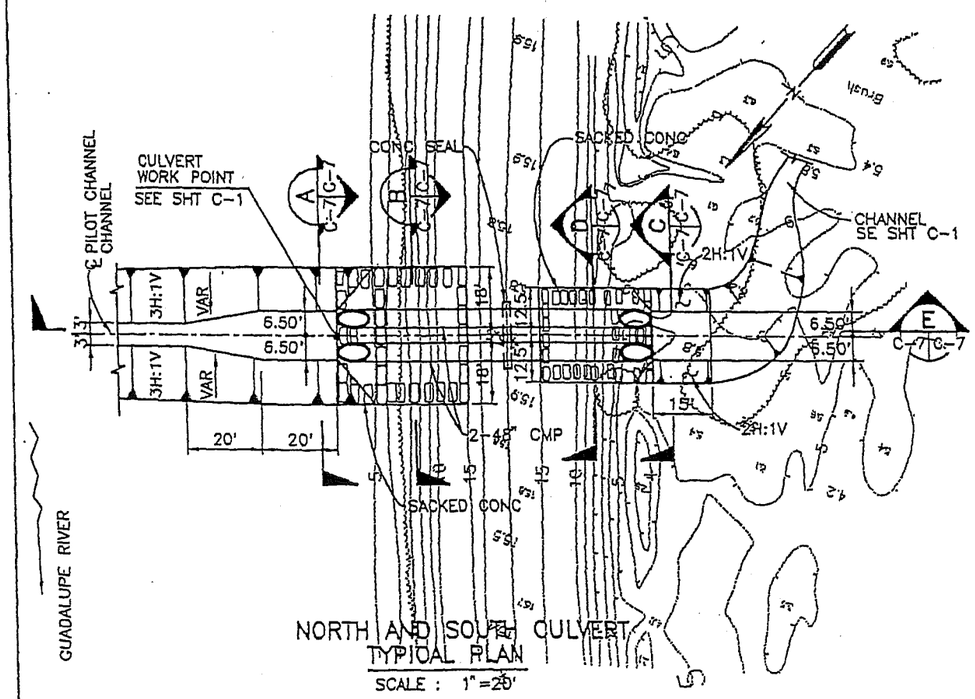
DETAIL 2
SCALE : 1/4"=1'-0"
C-7|C-7



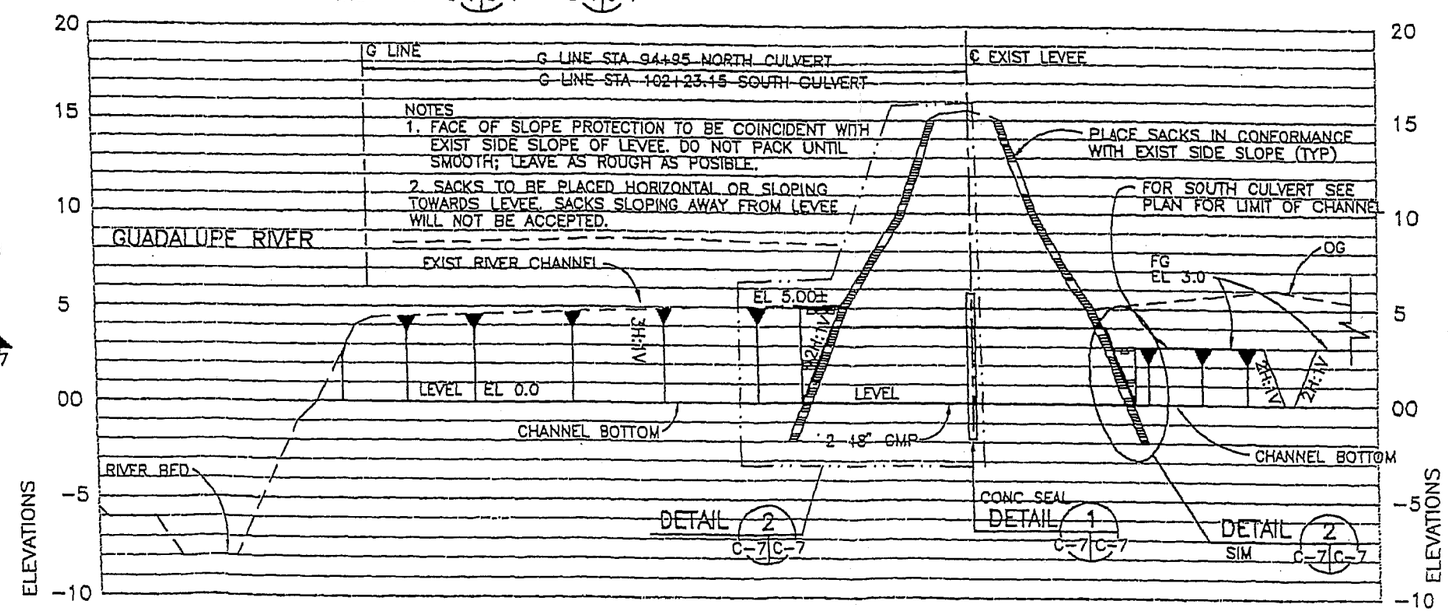
CONCRETE SEAL DETAIL 1
SCALE : 1/4"=1'-0"
C-7|C-7



SECTION B AND D
NTS
C-7|C-7 C-7|C-7



NORTH AND SOUTH CULVERT
TYPICAL PLAN
SCALE : 1"=20'



TYPICAL SECTION E
SCALE : HORIZ 1"=20'
VERT 1"=4'
C-1|C-7
C-7

4 0 4 8
1"=4'

20 0 20 40
1"=20'

2 0 2 4 6 8 10
1/4"=1'-0"

NO.	REVISIONS	DATE

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY

DESIGNED FRB	CADD FILE NAME 5413C07A	SUBMITTED	APPROVED
DRAWN FRB	CADD FILE DATE 12/31/96		
CHECKED GWB	PLOT DATE XXXXXX		
SCALE HORIZ. 1"=20' VERT 1"=4'	PCA NO. XXYY		

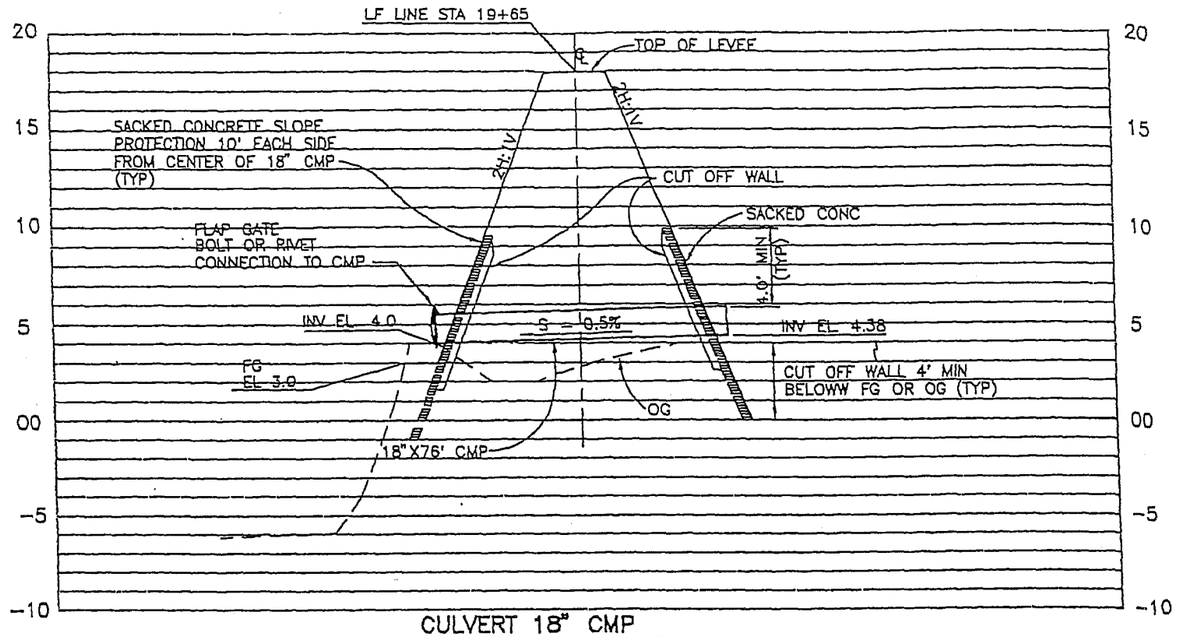
AN WEST INC.
Consulting Engineers
4123 Lakeside Drive
Richmond, CA 94806-1942

TASMAN CORRIDOR PROJECT
GUADALUPE MITIGATION SITE
PRELIMINARY
CULVERT
PLAN, SECTIONS AND DETAILS

SHEET XX OF XX
DRAWING NO. C-7
REVISION

BOND APPROVAL DATE XXXXXX | CONTRACT NO. XXXXXX | FILE NO. XXXXXX

PLOTTED: 01/30/97 11:47:04
FILE NAME: C:\PROJE...\5413\5413C07A



CULVERT 18" CMP
SECTION F
SCALE : HORIZ 1"=20' C-26 C-37
VERT 1"=4'

20 0 20 40
1"=20'
4 0 4 8
1"=4'

	REVISIONS	DATE

SANTA CLARA VALLEY TRANSPORTATION AUTHORITY

DESIGNED FRB	CADD FILE NAME 5413C08A	SUBMITTED	APPROVED
DRAWN FRB	CADD FILE DATE 12/31/96		
CHECKED GWB	PLOT DATE XXXXXX		
SCALE HORIZ. 1"=20' VERT 1"=4'	PCA NO. XXYY		

AMWEST INC.
Consulting Engineers
4123 Lakeside Drive
Richmond, CA 94806-1942

TASMAN CORRIDOR PROJECT
GUADALUPE MITIGATION SITE
SECTIONS AND DETAILS
PRELIMINARY

BOARD APPROVAL DATE XXXXXX	CONTRACT NO. XXXXXX	FILE NO. XXXXXX

SHEET XX OF XX
DRAWING NO. C-8
REVISION

FILE NAME: C:\PROJECTS\5413C08A PLOTTED: 01/30/97 11:45:58

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED LEVEE AND IMPROVEMENTS
GUADALUPE RIVER SITE
SAN JOSE, CALIFORNIA

SUBMITTED TO:

H.T. HARVEY & ASSOCIATES

906 ELIZABETH STREET

ALVISO, CALIFORNIA 95002

PREPARED BY:

GEO/RESOURCE CONSULTANTS, INC.

505 BEACH STREET

SAN FRANCISCO, CALIFORNIA 94133

JANUARY 1997

JOB NUMBER: 1935-000

Geo/Resource Consultants, Inc.
OLOGISTS ENGINEERS ENVIRONMENTAL SCIENTISTS

Corporate Headquarters
505 Beach Street
San Francisco, California 94133
(415) 775-3177 FAX (415) 775-2359
E-MAIL: GRC.INC@ix.netcom.com

ry 10, 1997
.000

Harvey & Associates
Elizabeth Street
.so, California 95002

: GEOTECHNICAL INVESTIGATION REPORT
PROPOSED LEVEE AND IMPROVEMENTS
GUADALUPE RIVER SITE
SAN JOSE, CALIFORNIA

:emen:

a smitted herewith is the report of our geotechnical investigation for the above referenced ct. Based on our investigation, we believe that the project is feasible from a geotechnical ngineering viewpoint, subject to the recommendations presented in the contents of this report.

/e reciate the opportunity to be of service to you for this project. If you have any questions egarding this report, please do not hesitate to contact us.

Respectfully submitted.
EO/RESOURCE CONSULTANTS, INC.

Alan D. Tryhorn, C.E.G
enior Vice President

Eric S. Ng, P.E., G.E.
Geotechnical Engineer

ADT/ESN:csc

TABLE OF CONTENTS

		PAGE
1.0	INTRODUCTION	1
2.0	PROJECT BACKGROUND	2
3.0	SCOPE OF WORK	3
4.0	FIELD EXPLORATION AND LABORATORY TESTING	4
4.1	FIELD EXPLORATION	4
4.2	LABORATORY TESTS	4
5.0	SITE CONDITIONS	6
5.1	SITE GEOLOGY AND SEISMICITY	6
5.2	SURFACE CONDITIONS	6
5.3	SUBSURFACE CONDITIONS	7
5.4	GROUNDWATER	8
6.0	DISCUSSION AND CONCLUSIONS	9
6.1	GENERAL GEOLOGIC AND SEISMIC RELATED HAZARDS	9
6.2	ON-SITE MATERIAL CONSIDERATIONS	10
6.3	PROPOSED LEVEE STABILITY EVALUATION	11
6.3.1	General Method	11
6.3.2	Loading Condition	11
6.3.3	Soil Parameters	11
6.3.4	Results of the analysis	12
6.4	PROPOSED LEVEE SETTLEMENT ESTIMATES	12
6.5	PROPOSED CULVERT INSTALLATIONS	12
7.0	RECOMMENDATIONS	14
7.1	LEVEE CONFIGURATION AND EROSION PROTECTION	14
7.1.1	Permanent Levee Slopes	14
7.1.2	Surface Drainage	14
7.2	EARTHWORK	14
7.2.1	Clearing and Subgrade Preparation	14
7.2.2	Material For Structural Fill	15
7.2.3	Compaction Requirement	15
7.2.4	Temporary Slopes and Excavations	15
7.3	CONSTRUCTION OBSERVATION AND TESTING	15
7.4	LEVEE MAINTENANCE AND SETTLEMENT MONITORING	16
7.5	ADDITIONAL SOIL ENGINEERING SERVICES	16

TABLE OF CONTENTS - continued

		PAGE
8.0	LIMITATIONS	17
9.0	REFERENCES	18

FIGURES

FIGURE 1	VICINITY MAP
FIGURE 2	SITE PLAN
FIGURES 3-10	BORING LOGS AND TEST PITS
FIGURE 11	SOIL CLASSIFICATION CHART AND KEY TO TEST DATA
FIGURES 12-13	CONSOLIDATION TEST REPORT
FIGURE 14	TRIAXIAL SHEAR TEST REPORT

APPENDIX

APPENDIX A	SLOPE STABILITY ANALYSIS

1.0 INTRODUCTION

Geo/Resource Consultants, Inc. (GRC) was retained by H. T. Harvey & Associates (H.T. Harvey) to conduct a geotechnical investigation for a proposed levee and two proposed culverts at the existing Guadalupe River levee locations. The site is located approximately 600 feet east of Gold Street and to the north of Alviso-Milpitas Road (Highway 237) in San Jose, California (see Figure 1 for the Site Location Map).

The proposed levee will be constructed east of the existing levee, in order to form the western barrier around a proposed 3.2 acre wetland mitigation area. We understand that the new levee will be approximately 1,250 feet long with a crest width of 16 feet, and constructed with 2h:1v (horizontal to vertical) side slopes up to elevation 18 (NGVD29). This datum (NGVD29) was established on the design plans by the civil engineer, AN West, Inc., and will be used throughout this report.

Presently, the mitigation area is covered with varying thicknesses of heterogenous fill. The fill consists primarily of soil, concrete rubble, steel rebar, rubber tires, household debris, and wood. The fill thickness varies from 0 feet at areas of previously undisturbed wetlands to as much as 10 feet in other areas. This fill was placed at the site over the past thirty to forty years. We understand that the mitigation area will be graded to approximately elevation +2.5 feet, with an ultimate design elevation of +3.5 feet. We have analyzed the stability of the proposed levee with the understanding that this fill will be removed to original ground along the length and width of the new levee construction. In our stability evaluation, our approach was based on criteria presented in the Federal Emergency Management Agency (FEMA) Revisions to National Flood Insurance Program Maps MT-2 Form 8, May 96, and 44 CFR Ch. 1, Section 65.10.

Two new culverts (48 inch diameter) will be placed at two locations through the existing Guadalupe River levee to allow river water to flow into and out of the proposed wetlands mitigation area. The invert of the new culverts will be at approximately elevation 0. A debris screen will be placed on the river side of the new culverts and rip rap may be placed surrounding the culverts. Shallow channels will be excavated at the two culvert locations both on the river side and the mitigation area to provide easier flow paths for the tidal waters. We understand these channels may also be excavated to approximately elevation 0 feet.

For this investigation, GRC was provided with a Site Plan prepared by AN West Inc., dated December 9, 1996. Design considerations for the new levee and culverts have been developed through recent conversations with Mr. Dan Stephens of H.T. Harvey Associates, Mr. Roy Molseed of the Santa Clara County Transportation Agency, Mr. Jerry Brownfield of AN West, Inc. Consulting Engineers, and Mr. Peter Sheydayi, P.E. of the Santa Clara Valley Water District (SCVWD).

2.0 PROJECT BACKGROUND

The Guadalupe River mitigation site was previously a wetland that has been covered with as much as 10 feet of rubble fill over more than 40 years. Material of indeterminate origin was placed over uplands, wetlands, and portions of the Guadalupe River. Based on the current plans, Santa Clara County is proposing to purchase the area and remove the fill. Approximately 3.2 acres will be purchased. To evaluate the suitability of the Guadalupe River Site for wetlands mitigation, several environmental site characteristic studies were previously performed by GRC. The scope included site visits, soil sampling, chemical and physical analysis of soil samples, and review of available environmental data. The results of the studies were presented in GRC's reports dated October 1995 and August 1996.

3.0 SCOPE OF WORK

As outlined in our proposal dated June 19, 1996, GRC's scope of work included the following:

- excavate and remove existing concrete and other assorted debris prior to drilling at the boring locations.
- provide subsurface condition information by drilling a total of 6 soil borings; two soil borings on the existing levee at the proposed culvert locations and 4 soil borings drilled along the proposed levee alignment.
- laboratory tests on soil samples including strength, moisture content, consolidation, and dry density, and other tests as deemed appropriate.
- evaluate site seismicity and seismic hazards at the levee site.
- provide design criteria for the proposed levee, including vertical and lateral stability, and estimates for differential and total settlements.
- provide design consideration for the culverts.
- provide site grading and excavation criteria for levee fill quality and compaction requirements.
- recommend subgrade preparation for the new levee construction.
- provide construction considerations and potential construction problems.
- provide a written soil report which will summarize the results of our investigation and our recommendations.
- review pertinent construction documents at 95% design for conformity with our recommendations.

Evaluation of the existing Guadalupe River levee is not included within this scope. A previous investigation of the existing levee was performed by others (Lowney, 1981).

4.0 FIELD EXPLORATION AND LABORATORY TESTING

The subsurface conditions of the site were explored on November 25 and 26, 1996, by drilling six exploratory borings (B-1 through B-6) to depths ranging from 25 feet to 35 feet below ground surface (bgs), and by excavating test pits (Test pits A and B).

4.1 FIELD EXPLORATION

Prior to initiating field activities, an encroachment permit was obtained from the owner of the property, Mr. Aldo J. Scaffone, and by the agency for the Guadalupe River levee, Santa Clara Valley Water District (SCVWD). A permit to drill exploratory borings was also obtained from the SCVWD (#96E000104). On November 20, 1996, two test pits (TP-A and TP-B) were excavated with a John Deere 690 excavator in the southern portion of the proposed levee to observe the extent of the concrete rubble. Three of the boring locations had to be cleared of up to 9 feet of concrete rubble and debris. This was accomplished with the excavator. The excavator dug a sloping ramp to a depth of approximately 9 feet, to provide drill rig access at the level of the native soils. Drilling was accomplished using a CME 75 truck mounted drill rig with 8-inch-diameter hollow-stem augers.

Our field engineer supervised the drilling and excavation operations, logged the soils encountered, and obtained soil samples for visual examination and laboratory testing. The approximate locations of the exploratory borings and test pits are presented on the Site Plan, Figure 2.

The samples were obtained with a 2.5-inch I.D., 3-inch O.D., Sprague and Henwood (S&H) sampler driven by an 140-pound automatic hammer falling 30 inches, and a 3-inch O.D. Shelby tube which was hydraulically pushed. Samples were generally collected and logged at 5-foot intervals. The descriptions of the soils encountered, sample depth and penetration resistance are presented on the boring and test pit logs in Figures 3 through 10. The soils are described in accordance with the Unified Soil Classification System shown in Figure 11.

Following drilling, each of the borings was filled with a cement grout mixture under the guidelines of the SCVWD drilling permit. The excavated test pits were backfilled.

4.2 LABORATORY TESTS

In our laboratory, the samples were re-examined to confirm their field classification and to select representative samples for testing. A testing program was established to evaluate the engineering properties of the site for design purposes. The testing program included moisture content, dry density determination, shear strength, and consolidation tests. The results of the laboratory tests are presented at corresponding locations on the boring logs as described by the Key to Test Data, shown in Figure 11.

The triaxial shear strength tests and consolidation tests were conducted by Cooper Testing Laboratory (CTL) of Mountain View, California. The triaxial shear strength and consolidation test results are presented on Figures 12 through 14.

5.0 SITE CONDITIONS

5.1 SITE GEOLOGY AND SEISMICITY

Lying on the margin of the San Francisco Bay plain, the project site is underlain by Holocene (less than 10,000 years old) estuarine and alluvial deposits. These soils include clays, silts and sands deposited by the Guadalupe River, which historically crossed portions of the site. The river, immediately east of the site, is presently contained within levees.

Bedrock beneath the site is very deep, and, based on nearby borings, is expected to be at least 700 feet deep.

The San Francisco Bay region is an area of high seismic activity. The San Andreas fault system, forming the bounding surface between the obliquely convergent North America and Pacific plates, trends northwest through the region in a band 50 mile wide. The San Andreas fault system has produced strong earthquakes in the past and may be expected to do so in the future.

Among the prominent fault zones within the San Andreas fault system are the Hayward, Calaveras, San Andreas (Northern), Las Positas, Sargent, Seal Cove-San Gregorio, and Greenville. A list of these major, active faults within the region, their maximum credible earthquake, and their distance from the site, follows:

Estimated Seismic Characteristics of Nearby Significant Faults

Fault	Distance and Direction from Site	Maximum Earthquake Magnitude	
	(miles)	Probable	Credible
Hayward	5 NE	6.5	7.5
Calaveras	9 NE	6.3	7.5
San Andreas (Northern)	13 SW	7.3	8.0
Las Positas	17 NE	4.4	6.3
Sargent	19 E	5.5	7.0
Seal Cove - San Gregorio	24 NW	6.8	7.7
Greenville	24 NE	5.2	7.3

In addition to active faults in the region, there are a number of inactive faults.

5.2 SURFACE CONDITIONS

The existing Guadalupe River levee runs northwest from highway 237 and creates the eastern boundary of this project. The levee slopes gently to the northwest from approximately elevation

16.5 feet at Station 103+69.86 to an intersection point with the proposed new levee at Station 90+80.79, elevation approximately 16.0 feet. The crest of the existing levee is approximately 12 feet wide and is typically sloped at 5h:3v on the river side and 2h:1v on the land side. The elevation of the toe of the existing levee on the river side is +4 to +5 feet and on the land side it is between -5 and +7 feet. The slopes of the levee are generally covered with wild brush and grasses. Minor settlement cracking was observed along much of the length of the levee running parallel to the crest.

At the location of the proposed levee, the majority of the area is covered with 6 to 10 feet of fill. The fill consists of concrete rubble, soil, asphalt, rubber tires, remnants of old piers, and household debris. The topography is very irregular and somewhat mounded with a low elevation of approximately +2 at the southern end and up to elevation +10 near the middle. The area is covered with ruderal vegetation.

Between the proposed levee and the existing levee is an area consisting of fill and remnants of the original Guadalupe River channel. The topography in this area is very irregular and mounded. Elevation varies from approximately -5 (former river channel) up to +12. The original ground elevation, prior to fill placement, was estimated at zero to -5 feet.

5.3 SUBSURFACE CONDITIONS

Beneath the 6 to 10 feet thickness of rubble fill, the subsurface conditions at the locations of the proposed levee, as revealed by our exploratory borings (B-1, B-2, B-5, and B-6) and test pits (TP-A and TP-B), consist of alluvial and marine deposits of sand, clays, and silts which were soft and loose to stiff. Borings B-1, B-2, and B-6 began at the elevation of the bottom of the concrete rubble (at an excavated depth of 7 to 9 feet below existing surface). Borings B1, B2, and B6 encountered alternating layers of stiff clays and sand. Boring B-5, located at the southern portion of the proposed levee, encountered approximately 8 feet of soft organic silts (Bay Mud) just below the 14.8 feet of existing road embankment fill. Underlying the Bay Mud, alternating layers of soft to stiff clays and sands were encountered.

Subsurface conditions at the two culvert locations, as revealed by our exploratory borings B-3 and B-4 at the existing levee, encountered approximately 16 feet of sandy clays and clay (compacted levee fill) underlain by 3 to 8 feet of clayey and sandy silts with organics (Bay Mud with some fine sand), and were soft to medium stiff. Plastic and glass were encountered at B-3 at 20 to 25 feet bgs, and plastic was encountered in B-4 at 18.5 feet bgs (boring B-4). This debris may have been pushed several feet into the soft soils prior to construction of the levee. Below the silts, the borings encountered a medium stiff clay (in boring B-3) and alternating layers of sandy clays and clayey sands which were medium stiff and medium dense to approximately 24 feet bgs (boring B-4). Below 24 feet, the borings encountered medium stiff clay and clayey silt.

Two test pits were excavated along the southern portion of the proposed levee. Subsurface conditions, as revealed by our exploratory test pit TP-A, consisted of 2.5 feet of fill, largely silty sandy clay with some rock, overlying soft Bay Mud to the depth explored (5 feet). At test pit TP-B, we encountered 4 feet of fill consisting of sandy silt with concrete, steel rebar, traces of brick, and wood. The fill was underlain by medium stiff Bay Mud to approximately 6 feet bgs, which is further underlain by brown and green brown silty clay which was medium stiff to the depth explored (8 feet).

Prior to this investigation, GRC had also conducted limited field exploration for environmental site characterization. The results of the previous investigation are incorporated into this evaluation.

In the previous test-pit excavations, the soils (predominantly fill) consisted of brown sandy silts, gravelly sands, and sandy clays with varying quantities of concrete rubble and debris. To the depths explored (2 to 7 bgs) the fill was loose, dry to moist, and contained an estimated 10% to 75% concrete rubble. The concrete rubble, much of it with steel reinforcement, included what appeared to be debris from the demolition of walkways and concrete paving areas. Asphalt, metal, household debris, tires, and wood were also observed in minor quantities throughout the site. In one test pit located near the eastern limit of the proposed site, where the old Guadalupe River channel once flowed the remains of a wooden pier were encountered and a strong creosote-type odor was noted.. In other test pits where the native/fill interface was encountered, below the fill was a dark brown, gray/black clay with organics (the original wetlands soils).

5.4 GROUNDWATER

We encountered groundwater during drilling activities at the following approximate elevations: B-1 (-5.5), B-2 (-6), B-3 (-4), B-5 (-5.5), and B-6 (-4). We did not encounter groundwater in boring B-4. The groundwater levels observed were not stabilized, and because of the nature of drilling in clayey soils, may not represent typical or long-term conditions. Higher groundwater should be anticipated. Fluctuations in the level of groundwater and flow direction can occur due to variations in rainfall, tides, and other factors.

6.0 DISCUSSION AND CONCLUSIONS

In our opinion, from a geotechnical viewpoint, the site is suitable for the proposed improvements, provided the recommendations presented in this report are followed during design and construction. The primary geotechnical concern is the presence of rubble fill and soft clay conditions at the levee foundation level. Because of permeability requirements and environmental concern of the on-site materials, we have based our levee analysis on the assumption that all the rubble fill will be removed from site and will not be considered in the levee design. The presence of the soft soils requires geotechnical consideration of levee stability under various loading conditions and future settlement due to the weight of levee fill. Both considerations have been incorporated in the design.

6.1 GENERAL GEOLOGIC AND SEISMIC RELATED HAZARDS

Seismic impacts to the levee as a result of earthquakes may be either primary or secondary in nature. Primary effects include fault rupture, ground shaking, subsidence and/or uplift. Secondary impacts may be induced by primary effects, and include liquefaction, ground failure (lurching, lateral spreading, slope failure), and seiche. These potential hazards are discussed below:

1. Fault rupture.
   According to maps published by the state of California (DMG 1982, 1994), the proposed levee is not crossed by faults considered by the State to be active. The potential impact from fault rupture is low.
2. Ground shaking.
   The proposed levee is approximately 5 miles from the active Hayward fault and is in close proximity to other active faults in the Bay Area. The potential for strong ground shaking from future earthquakes generated on local or regional faults is a significant seismic hazard. During the life of the project, moderate to strong ground shaking should be anticipated.
3. Subsidence and/or uplift
   Such phenomena are generally restricted to areas immediately adjacent to zones of fault rupture. There are no known active faults underlying the site, the potential for subsidence and/or uplift is considered to be low.
4. Liquefaction
   Liquefaction phenomena can occur in loose, saturated granular soils during seismic shaking when pore water pressure build-up reaches the confining pressures of the soils and subsequently causes a reduction in the soil strength. An evaluation of liquefaction potential throughout San Jose was conducted by Consultants in May, 1992 (Power, and others, 1992). That study locates the site in an area of high liquefaction potential due to shallow groundwater conditions; however, that study was regional in nature, rather than site specific, and did not consider the characteristics of the local cohesive soils. Soils at the proposed levee site are clays, clayey sands, and clayey silts. In general, cohesive soils that contain

more than about 20 percent clay are less susceptible to liquefaction (Seed and Idriss, 1982). The potential of soil liquefaction is considered to be low.

5. Ground failure (differential settlement, lurching, spreading, densification).
   Strong ground shaking is expected at the levee during an earthquake. The soils underlying the levee are soft to stiff. The potential for seismically-induced ground failure is considered to be moderate.
6. Seismically-induced slope failure.
   The site is located on low terrain along the bay margins. The potential for impact to the site from seismically-induced slope failure is low.
7. Seiche.
   Seiches, or standing waves, produced in an elongate body of water, present a seismic hazard to the levee. The potential for erosion from overtopping of the levee as a result of seiche action or flooding following a major earthquake should be evaluated for the levee's long term condition.

6.2 ON-SITE MATERIAL CONSIDERATIONS

GRC has previously performed a limited environmental site characterization at the subject site in both October, 1995, and August, 1996. Results of chemical analyses of on-site fill soil samples indicate the presence of TRPH, chlordane, toluene, phthalates, and individual metals. The concentrations detected in these samples, however, are below concentrations which would characterize them as hazardous waste.

The on-site fill material generally would be designated as either Class II or Class III, based on sampling performed to date. If some of this material were to be left in place and/or incorporated into the proposed levee, it is probable that at some locations, the concentrations (i.e., mercury, and lead) in the fill materials will exceed the Regional Water Quality Control Board (RWQCB) sediment screening criteria (SSC) for cover or noncover material.

GRC prepared a report entitled "Wetlands Mitigation Soil Disposal, Options Evaluation and Cost Analysis", for this site in August, 1996. A comparison of alternatives for the on-site fill material concluded that the optimal solution would be removal of fill material. That alternative proposed to segregate the soil and debris from the concrete rubble, with the soil and debris hauled to a Class II or Class III landfill and the concrete hauled to a concrete recycler.

Field observations indicate that the fill material consists of large pieces of concrete, rebar, rubber tires, organics, and other debris unsuitable for use as engineered fill in construction of the proposed levee. On December 3, 1996, GRC had discussions with SCVWD (Peter Sheydahi, personal communications) regarding the possible use of the fill material in the proposed levee. The indication from SCVWD was this material would have a problem meeting some of the levee design criteria such as permeability and contamination issues. Therefore, utilizing the rubble fill in the construction of the levee was not further considered.

6.3 PROPOSED LEVEE STABILITY EVALUATION

6.3.1 General Method

The stability of the proposed levee embankment was analyzed using the simplified Bishop's method of slices with the aid of a computer program (Geosystem Slope Stability Program). The levee was configured to be 18 feet high on the water side and 13 feet on the land side representing the general surface elevation. The embankment is assumed to be 16 feet wide at the crest with both side slopes at 2 horizontal to 1 vertical. Typical sections are presented in Appendix A.

6.3.2 Loading Condition

The proposed levee was analyzed under several loading conditions as listed below.

Case 1: Static loading, end of construction
Case 2: Static loading, sudden drawdown
Case 3: Static loading, critical flood stage
Case 4: Earthquake loading

Under normal condition, the groundwater level is assumed to be at elevation +2 feet. During flooding stage and sudden drawdown conditions, the water is assumed to be at elevation +15 feet. During earthquake loading, a lateral coefficient of 0.15 was selected in the analysis to represent the seismic forces acting on the embankment.

In cases 1, 2, and 4, the embankment on the water side was regarded as the most critical section and was selected for analysis. In case 3, the embankment on the land side was deemed to be most critical and used in the analysis.

6.3.3 Soil Parameters

The soil parameters used in the analysis were derived from results of our laboratory testing program and also review of previous data from our files and similar soil data from other investigations. Both effective stress and total stress soil parameters were used. For long term static condition, effective soil parameters were used to represent a drained condition. In other loading conditions, the total stress parameters were used to represent an undrained condition.

The proposed levee is assumed to be constructed of compacted engineered fill, consisting of clayey materials. The suitability of the import clayey fill should be further evaluated by GRC. The foundation materials consist of soft to medium stiff Bay Mud. Based on data that we have

gathered and similar experience with this type of soil in other projects (Mitchell, and others, 1992), the following average strength parameters were used in the analysis:

	Effective Stress		Total Stress	
	Friction Angle	Cohesion	Friction Angle	Cohesion
	(degrees)	(psf)	(degrees)	(psf)
Embankment	30	200	0	1300
Foundation	19	0	0	550

6.3.4 Results of the analysis

Based on the criteria and loading condition as stated above, the results of the stability analysis are presented below:

Case	Loading Condition	Factor of Safety
1	Static Long Term	1.43
2	Static Rapid drawdown	1.3
3	Static flooding stage	3.0
4	Earthquake loading	1.15

The cases analyzed met the minimum safety factors as needed in the design. Details of the calculations are presented in Appendix A of this report.

6.4 PROPOSED LEVEE SETTLEMENT ESTIMATES

Based on the foundation condition of the proposed levee site, we anticipate that the levee embankment will go through long term settlement resulting from the consolidation of Bay Mud stratum. Based on the consolidation test results and the estimated weight of the overburden levee fills, we estimate that the levee will have total settlement ranging from 9 inches to a maximum of 3.6 feet. The majority of the settlement will occur within 25 years after construction. The maximum settlement will occur at the southern east-west section of the new levee and at areas where the original Guadalupe River crosses the new alignment, where the Bay Mud would be the thickest. In most of the levee alignment, however, we anticipate that the total settlement to be on the order of 1 foot. Because of the settlement issue, it will be necessary to maintain the crest elevation and to repair settlement cracks and depressions from time to time.

6.5 PROPOSED CULVERT INSTALLATIONS

Previous investigation by Lowney and Associates (1981) has indicated that future settlement of the existing levee is likely to continue over a period of time. The culverts proposed to be placed within the existing levee are likely to undergo similar settlement. GRC understands that

additional flow capacity will be incorporated into the culvert design to allow for capacity reduction due to settlement.

7.0 RECOMMENDATIONS

7.1 LEVEE CONFIGURATION AND EROSION PROTECTION

7.1.1 Permanent Levee Slopes

The proposed levee should be constructed over competent native soils. We recommend that the planned permanent levee slopes be filled to an inclination no steeper than 2:1 (horizontal to vertical). Exposed slopes may be subject to minor sloughing, settlement cracks, animal burrows and erosion which may require periodic maintenance. We recommend that these slopes be planted soon after construction to minimize erosion potential, and should be monitored over the long-term for detrimental effects of erosion and burrowing animal activity.

7.1.2 Surface Drainage

The finish grades should be designed to drain surface water away from the levee surface and slope areas, to suitable discharge points. Slopes of at least 1 percent are recommended. Ponding of water on levee surfaces should not be allowed.

7.2 EARTHWORK

7.2.1 Clearing and Subgrade Preparation

All deleterious materials, including concrete rubble, asphalt concrete, rubber tires, household debris, roots, vegetation etc., found in the site fill and native soils should be cleared from the site and adjacent areas, which will be reworked to receive structural fill.

At the southern portion of the new levee, where the new levee will be placed against an existing highway embankment, the surface should be stripped of all deleterious materials. The actual stripping depth required should be established by us at the time of construction.

After the site has been properly cleared, stripped, and excavated to the required grades, the exposed surface soil in areas to receive structural fill should be scarified to a depth of 6-inches, moisture conditioned, and compacted to the specifications for structural fill, listed below under the section captioned "compaction" requirement. Where soft soils are encountered as foundation material, overexcavation and/or a geo-fabric may be necessary. GRC should be present at the site to observe the exposed subgrade prior to placement of fill.

For any fill placed on slopes having an inclination steeper than 6 horizontal to 1 vertical, we should be contacted for keying and benching criteria. At the southern portion of the new levee, new levee fill should follow this recommendation.

7.2.2 Material For Structural Fill

Import material for the new levee fill containing less than 3 percent organic material by volume (ASTM D2974) is suitable for use as structural fill. However, structural fill placed at the site should not contain rocks or pieces larger than 6 inches in greatest dimension, and contain no more than 15 percent larger than 2.5 inches. Material for fill should be predominantly a clay, classified in the Unified Soil Classification as a fine grained soil "CL" (low to medium plasticity clay) with a plasticity index of less than 18 percent. The proposed fill should contain no more than 25% by weight of granular soils (retained by 200 sieve). Our representative should approve fill materials prior to their use on-site.

7.2.3 Compaction Requirement

The scarified surface soils and all structural fill should be compacted in uniform lifts, no thicker than 8 inches in uncompacted thickness, conditioned to the appropriate moisture content, and compacted to 95% relative to ASTM D 1557, latest edition.

7.2.4 Temporary Slopes and Excavations

The contractor should be responsible for the design and construction of all temporary slopes and any required shoring. Shoring and bracing should be provided in accordance with all applicable local, state and federal safety regulations, including the current OSHA excavation and trench safety standards. Those excavations less than 5 feet high in competent material where water is not present may be cut vertical. Temporary slopes higher than 5 feet should be shored, or cut back to an inclination of at least 1:1 (horizontal to vertical), as determined by the contractor.

Excavations which extend below finish grade should be backfilled with structural fill, as described above.

7.3 CONSTRUCTION OBSERVATION AND TESTING

The earthwork and foundation phases of construction should be observed and tested by GRC to 1) establish that subsurface conditions are compatible with those used in the analysis and design; 2) observe compliance with the design concepts, specifications and recommendations; and 3) allow design changes in the event that subsurface conditions differ from those anticipated. The recommendations in this report are based on limited number of borings. The nature and extent of variation across the site may not become evident until construction. If variations are then exposed, it will be necessary to reevaluate our recommendations.

7.4 LEVEE MAINTENANCE AND SETTLEMENT MONITORING

We estimate that the levee will have total settlement ranging from 9 inches to a maximum of 3.6 feet. In most of the levee alignment, however, we anticipate that the total settlement to be on the order of 1 foot. Because of the settlement issue, it will be necessary to maintain the crest elevation and to repair settlement cracks and depressions from time to time. Periodic inspection of levee should be performed to evaluate the condition of the levee surface and the structural integrity of the levee embankment.

7.5 ADDITIONAL SOIL ENGINEERING SERVICES

Prior to construction, we should review the final grade and foundation plans and specifications to verify conformance with the intent of our recommendations. We should also approve proposed levee fill materials. In the event that changes occur in the nature, design, or location of structures, the conclusions and recommendations presented in this report shall not be considered valid unless the changes are reviewed by us and the conclusions either verified or modified as required. To a degree, the performance of the new construction is dependent on the procedure and quality of construction. Therefore, we recommend that we provide on-site observations of the contractor's procedures and the exposed soil conditions together with field and laboratory testing during site preparation and grading, placement and compaction of fill and backfill, and foundation construction. These observations will allow us to correlate subsurface conditions exposed during excavation with those evaluated in our investigation, and make necessary changes in the foundation support and other geotechnical design criteria, if necessary.

8.0 LIMITATIONS

This report is based upon the services we provided in conducting the Geotechnical Investigation for HT. Harvey and Associates, as described herein. The scope of services associated with this report was developed specifically for H.T. Harvey in light of their risk management preferences.

Note, too, that the report is subject to certain limitations that may or may not be noted in the report itself. In addition, recognize that the passage of time affects the information provided in the report: Our opinions relating to site conditions are based upon information that existed at the time our conclusions were formulated. As we are sure you can appreciate, site conditions can change rapidly, such as seasonally, or in some cases, overnight.

Please understand that the services provided for in this project were limited to the specific requirements of H.T. Harvey; the limited scope of service allows our firm to form no more than an opinion of the actual conditions at the site. This letter confers upon no third party other than the design team the right to rely upon the information contained in the report or this letter. No other party is entitled to rely on the report unless our express written consent is first obtained.

9.0 REFERENCES

California Department of Conservation, Division of Mines and Geology (DMG). 1982. Alquist Priolo Earthquake Fault Map, Milpitas Quadrangle.

California Department of Conservation, Division of Mines and Geology (DMG), 1994. Special Publication 42.

Federal Emergency Management Agency, "Revisions to National Flood Insurance Program Maps, Application Form 8, May 1996.

Geo/Resource Consultants, Inc., "Wetlands Mitigation Soil Disposal, Options Evaluation and Cost Analysis Report. Guadalupe River. San Jose, California", August 1996.

Geo/Resource Consultants. Inc.. "Wetlands Screening. Guadalupe River Site. San Jose. California". August 1996.

Geo/Resource Consultants. Inc.. "Site Characterization Report. Guadalupe River Site, San Jose, California". October 1995.

Lowney, John V. & Associates, "Geotechnical Investigation. Guadalupe River Improvements", January 23. 1981.

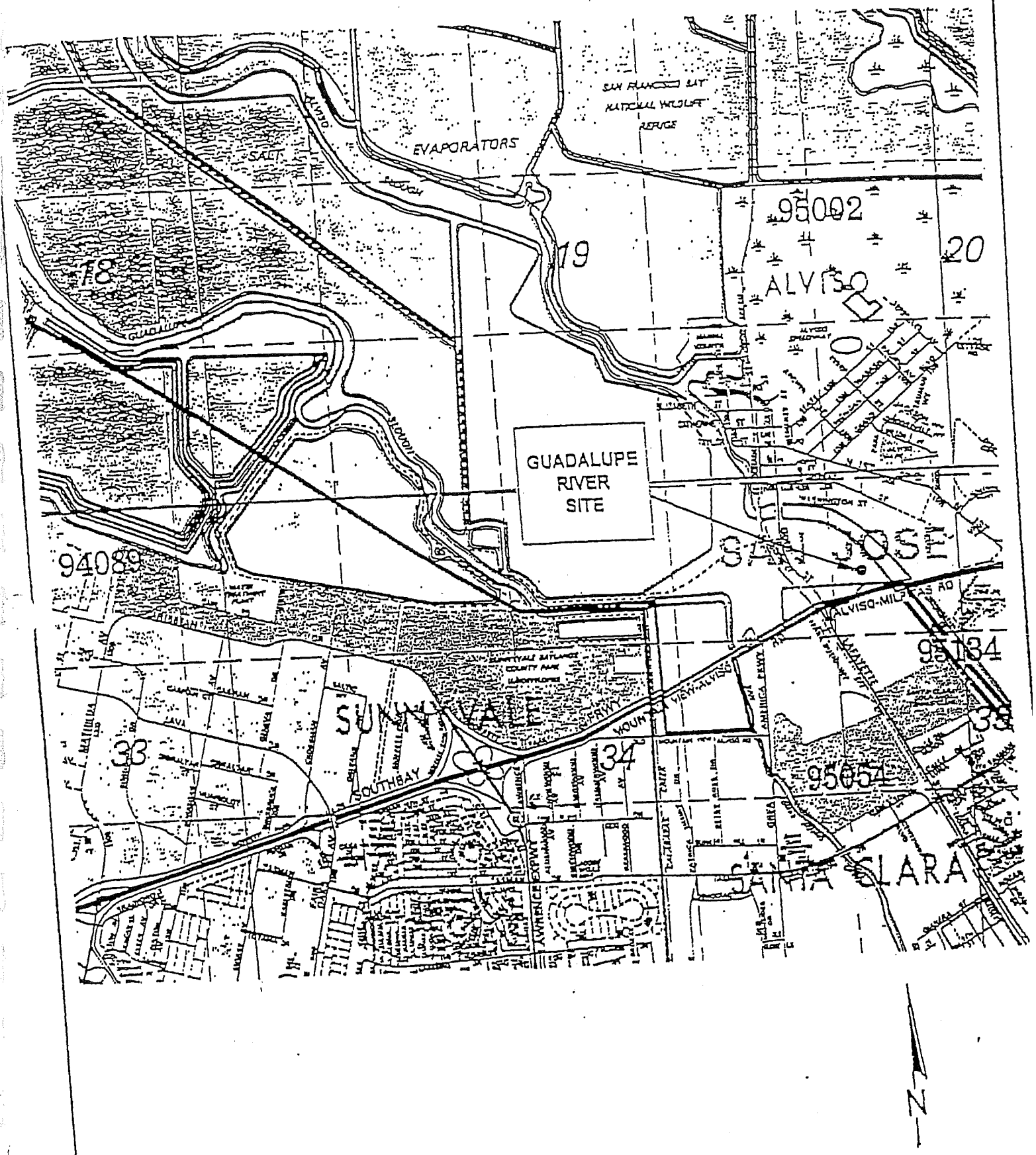
Mitchell, R.A.. Hatch. S.. and Siegel. R, 1992 "Stability and Closure Design for a Landfill on Soft Clay and Peat". Proceedings on Stability and Performance of Slopes and Embankments. - II. ASCE Geotechnical Special Publication 31, June-July. 1992.

National Flood Insurance Program Regulations, "44 CFR Ch. 1, Section 65.10. Mapping of Areas Protected by Levee Systems", August 25, 1986.

Power, Maurice S.. Wesling, John R., Perman. Roseanne C. Youngs. Robert R.. and DiSilvestro, Laurel A. 1992. Evaluation of Liquefaction Potential in San Jose. California. Final Technical Report. May 1992. Geomatrix Consultants.

Seed, H.B., and Idriss. I.M.. 1982. Ground motions and soil liquefaction during earthquakes: Earthquake Engineering Research Institute monograph, v.5 in a series of Engineering Monographs on Earthquake Criteria, Structural Design and Strong Motion Records.

FIGURES



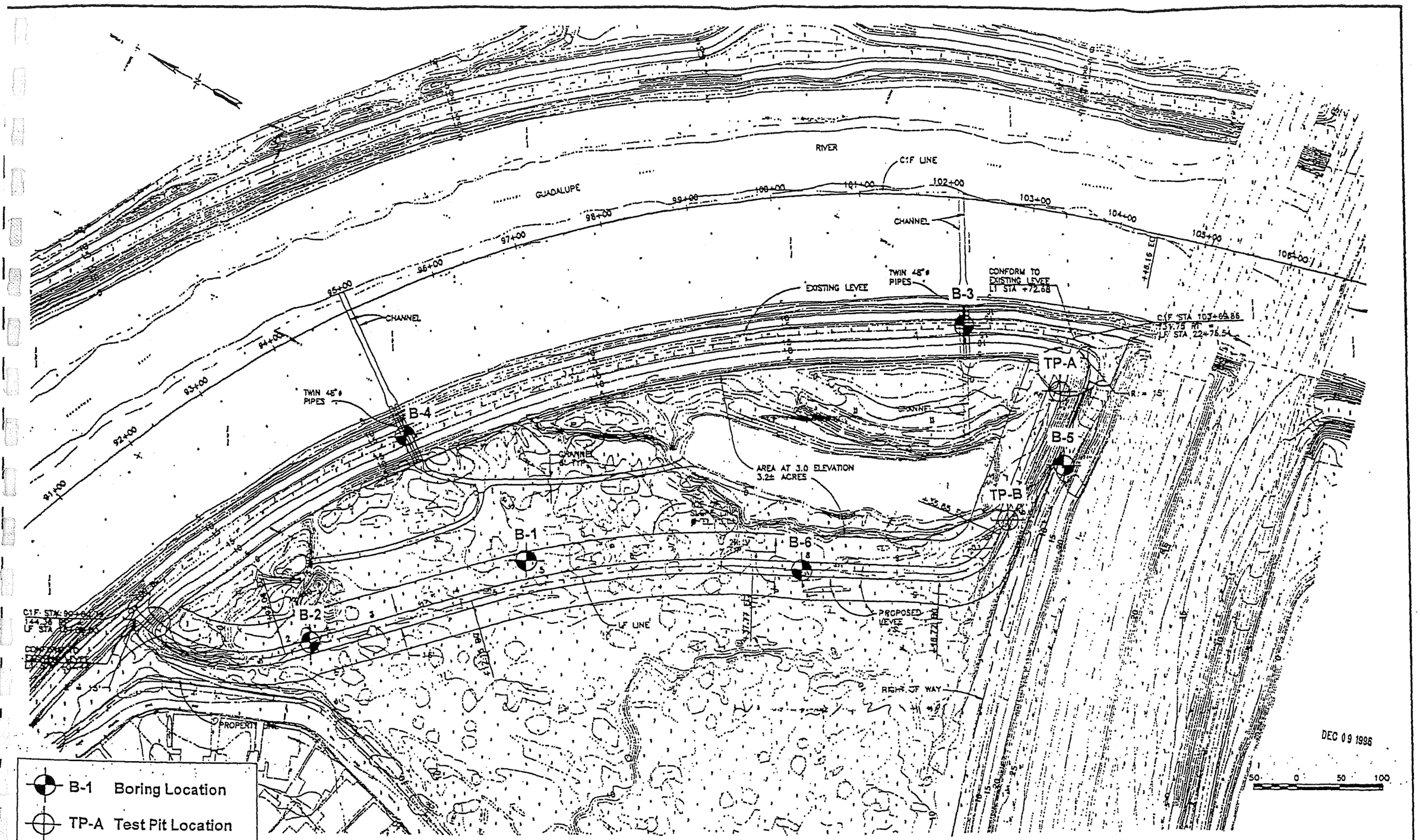
Reference: Thomas Guide, 1989

Geo/ Resource Consultants, Inc.
GEOLOGISTS / ENGINEERS / ENVIRONMENTAL SCIENTISTS
505 BEACH STREET, SAN FRANCISCO, CALIFORNIA 94133

LOCATION MAP

TASMAN CORRIDOR PROJECT
GUADALUPE RIVER SITE

FIGU
1



SANTA CLARA COUNTY TRANSPORTATION AGENCY			TASMAN CORRIDOR PROJECT GUADALUPE MITIGATION SITE	C-1
DESIGNED	5413CT	SUBCONSULTANT LOGO	CONSULTANT LOGO	
DRAWN	10/01/96			
CHECKED				
1"=50'				

Geo/ Resource Consultants, Inc.
GEOLOGISTS / ENGINEERS / ENVIRONMENTAL SCIENTISTS
906 BEACH STREET, SAN FRANCISCO, CALIFORNIA 94133

Job No. 1935-000 Appr. AOT Date 12/96

SITE PLAN

TASMAN CORRIDOR PROJECT
GUADALUPE RIVER SITE
SAN JOSE, CALIFORNIA

FIGURE 2

LOG OF BORING B-1

Equipment CME-75

Elevation -2.5' ± Date 11/25/96

boratory Analysis	Blows/ft.	Moisture Content (%)	Dry Density (pcf)	Depth (ft.)	Sample pnts.	
				0		DARK BROWN SILTY CLAY (CH) medium stiff, wet
	29	22.0				▼ color change to mottled olive brown and yellow brown, trace coarse sand, medium stiff
				5		
	22	26.3				grades sandy
				10		MOTTLED GREEN BROWN AND YELLOW BROWN SILTY SANDY CLAY (CL) medium stiff
	21	19.6				
				15		Alternating layers (±6") of BROWN CLAYEY SAND (SC) loose to medium dense and BROWN SANDY CLAY (CL) soft to stiff
	16	20.7	110.9			
				20		
	9	23.1				
				25		
	33	19.1	111.6			
				30		Boring terminated at 26.5' Groundwater encountered at 3.5' at time of drillir

GeolResource Consultants, Inc.
Geologists / Engineers / Environmental Scientists

Job No. 1935-000 Appr: AOT Date 12/96

LOG OF BORING B-1
TASMAN FINAL
WETLANDS MITIGATION
GUADALUPE RIVER SITE
SAN JOSE, CALIFORNIA

FIGU
3

LOG OF BORING B-2

Equipment CME-75

Elevation -1.0' ± Date 11/25/96

Laboratory Analysis	Blows/ft.	Moisture Content (%)	Dry Density (pcf)	Depth (ft.)	Sample pnts.	Description
				0		BROWN SANDY CLAY (CL) with rubble
	10	23.8				DARK BROWN SANDY CLAY/CLAYEY-SAND (CL/SC) soft to medium, wet
	20	24.7		5		▼ DARK GREY SILTY CLAY (CH) stiff, saturated, partially decomposed wood @ 5.5'
TXUU 1500 (1400)	650psi push	19.6	110.2	10		YELLOW BROWN SANDY CLAY (CL) trace coarse sand, slightly mottled, medium stiff
				15		grades sandy
	47	17.7	117.7	20		BROWN CLAYEY SAND (SC) dense, medium to coarse grain, saturated
	75	16.2		25		BROWN SAND (SW) very dense, coarse grain
				30		Boring terminated at 25.0' Groundwater encountered at 5.0' at time of drillin

GeoIResource Consultants, Inc.
Geologists / Engineers / Environmental Scientists

Job No. 1935-000 Appr: AOT Date 12/96

LOG OF BORING B-2

TASMAN FINAL
WETLANDS MITIGATION
GUADALUPE RIVER SITE
SAN JOSE, CALIFORNIA

FIGUR 4

LOG OF BORING B-3

Equipment: CME-75

Elevation: 16.5' ± Date: 11/26/96

Laboratory Analysis	Blows/ft.	Moisture Content (%)	Dry Density (pcf)	Depth (ft.)	Sample pnts.	Description
				0		DARK BROWN SANDY CLAY (CL)
				5		DARK BROWN CLAY (CH) stiff, moist, geofabric at 6.0'
	29					LEVEE FILL
				10		
	11	20.7		15		
						BLUE GREY SANDY SILT (ML-MH) Bay Mud, very fine sand, soft to medium stiff
Consolidation	500 psi push	35.4	85.5	20		debris, glass at 19.5'-20.0'
TXUU 613 (2700)	11	34.4 40.9	87.6			GREY BROWN CLAY (CH) Bay Mud, medium stiff, some glass and plastic
				25		Boring terminated at 25.0' Groundwater encountered at 19.5' at time of drill
				30		

GeoResource Consultants, Inc.
Geologists / Engineers / Environmental Scientists

Job No. 1935-000 Appr: AOT Date 12/96

LOG OF BORING B-3
TASMAN FINAL
WETLANDS MITIGATION
GUADALUPE RIVER SITE
SAN JOSE, CALIFORNIA

FIG

LOG OF BORING B-4

Equipment: CME-75

Elevation: 16.0' ± Date: 11/26/96

Laboratory Analysis	Blows/ft.	Moisture Content (%)	Dry Density (pcf)	Depth (ft.)	Sample pnts.	Description
				0		DARK BROWN SANDY CLAY (CL) moist
				5		
	30	21.1	108.7			DARK BROWN CLAY (CH) very stiff, moist
				10		grey brown
						softer at 12' to 13' glass fragments
				15		
	9	25.4	14.0			MOTTLED BLUE GREY AND DARK BROWN CLAYEY SILT (OH) organics, soft to medium stiff, wet
	13	27.3		20		YELLOW BROWN AND BLUE GREY SANDY CLAY (CL) medium stiff and CLAYEY SAND (SC) Alternating layers (± 6") medium dense, wet, plastic encountered at 18.5'
	17	25.0		25		DARK BROWN CLAYEY SILT (MH) medium dense, wet to saturated
				30		Boring terminated at 25.0' no groundwater encountered

LEVEE FILL

Geo/Resource Consultants, Inc.
Geologists / Engineers / Environmental Scientists

Job No. 1935-000 Appr: AMT Date 12/96

LOG OF BORING B-4
TASMAN FINAL
WETLANDS MITIGATION
GUADALUPE RIVER SITE
SAN JOSE, CALIFORNIA

FIGU

LOG OF BORING B-5

Equipment CME-75

Elevation 10.5' ± Date 11/26/96

Laboratory Analyses	Blows/Foot	Moisture Content (%)	Dry density (pcf)
	32	15.4	120.6
	42	12.7	123
	24		
TXUU 550 (2300)	7	44.4	76.9
Consolidation	300-700 psi push	23.1 24.8	100.6 101.8
	10	26.3	
	31	23.0	

Depth (ft.) 0, 5, 10, 15, 20, 25, 30, 35, 40

Sample pnts.

BROWN SANDY CLAY (CL)
with gravel, very stiff, moist

LEVEE FILL

DARK GREY SANDY GRAVEL (GW)
dense, moist, contains asphalt concrete

DARK GREY BROWN TO BLACK CLAYEY SILT (MH)
Bay Mud, soft, saturated

Alternating layers (±6") of
YELLOW BROWN CLAY (CH)
trace sand, stiff
and
CLAYEY SAND (SC)
loose

Boring terminated at 35.0'

Groundwater encountered at 16.0' at time of drillin

Geo/Resource Consultants, Inc.
Geologists / Engineers / Environmental Scientists

Job No. 1935-000 Appr: AOT Date 12/96

LOG OF BORING B-5
TASMAN FINAL
WETLANDS MITIGATION
GUADALUPE RIVER SITE
SAN JOSE, CALIFORNIA

FIGU
7

LOG OF BORING B-6

Equipment CME-75

Elevation -3.0' ± Date 11/26/96

Laboratory Analysis	Blows/ft.	Moisture Content (%)	Dry Density (pcf)	Depth (ft.)	Sample pnts.	
				0		DARK BROWN CLAYEY SAND (SC) loose, wet to saturated
	20	25.1 20.6	109.6			
				5		MOTTLED OLIVE BROWN AND YELLOW BROWN CLAY (CH) trace fine sand, medium stiff
	11	19.0				
				10		BROWN CLAY (CH) stiff
						BROWN CLAYEY SAND (SC) medium dense
	26	19.7	113.1			
				15		BROWN CLAY (CH) trace sand, slightly mottled grey brown, stiff
						BROWN SANDY SILT (ML) medium stiff
	8	23.7				
				20		grades clayey
						BROWN CLAYEY SILT (MH) medium stiff
	14	23.6	105.8			
				25		Boring terminated at 25.0'
						Groundwater encountered at 5.0' at time of drillir
				30		

GeoIResource Consultants, Inc.
Geologists / Engineers / Environmental Scientists

Job No. 1935-000 Appr: AOT Date 12/96

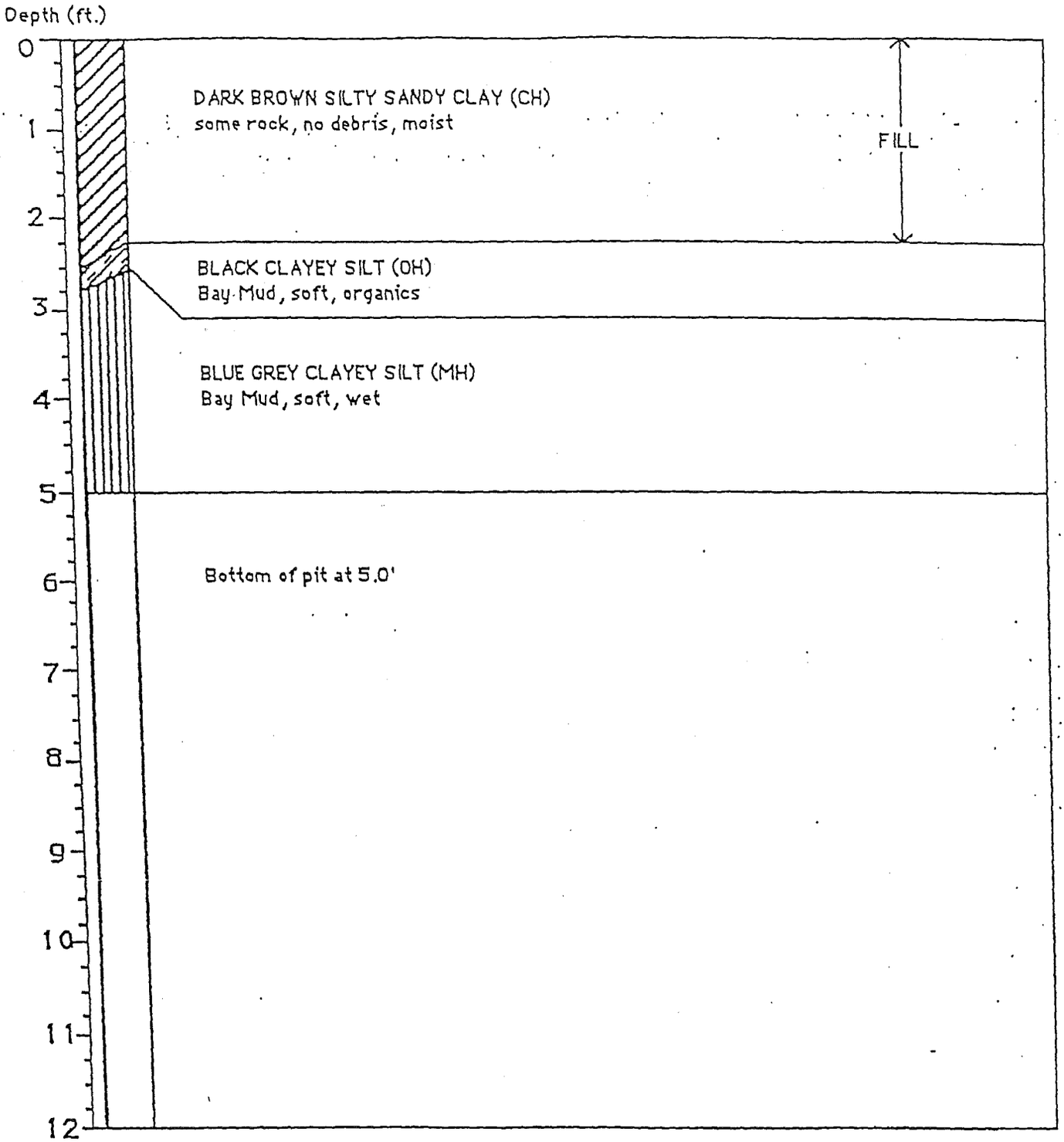
LOG OF BORING B-6
TASMAN FINAL
WETLANDS MITIGATION
GUADALUPE RIVER SITE
SAN JOSE, CALIFORNIA

FIG

LOG OF TEST PIT TP-A

Equipment: John Deere 690 Excavator

Elevation: 1.3' ± Date: 11/20/96

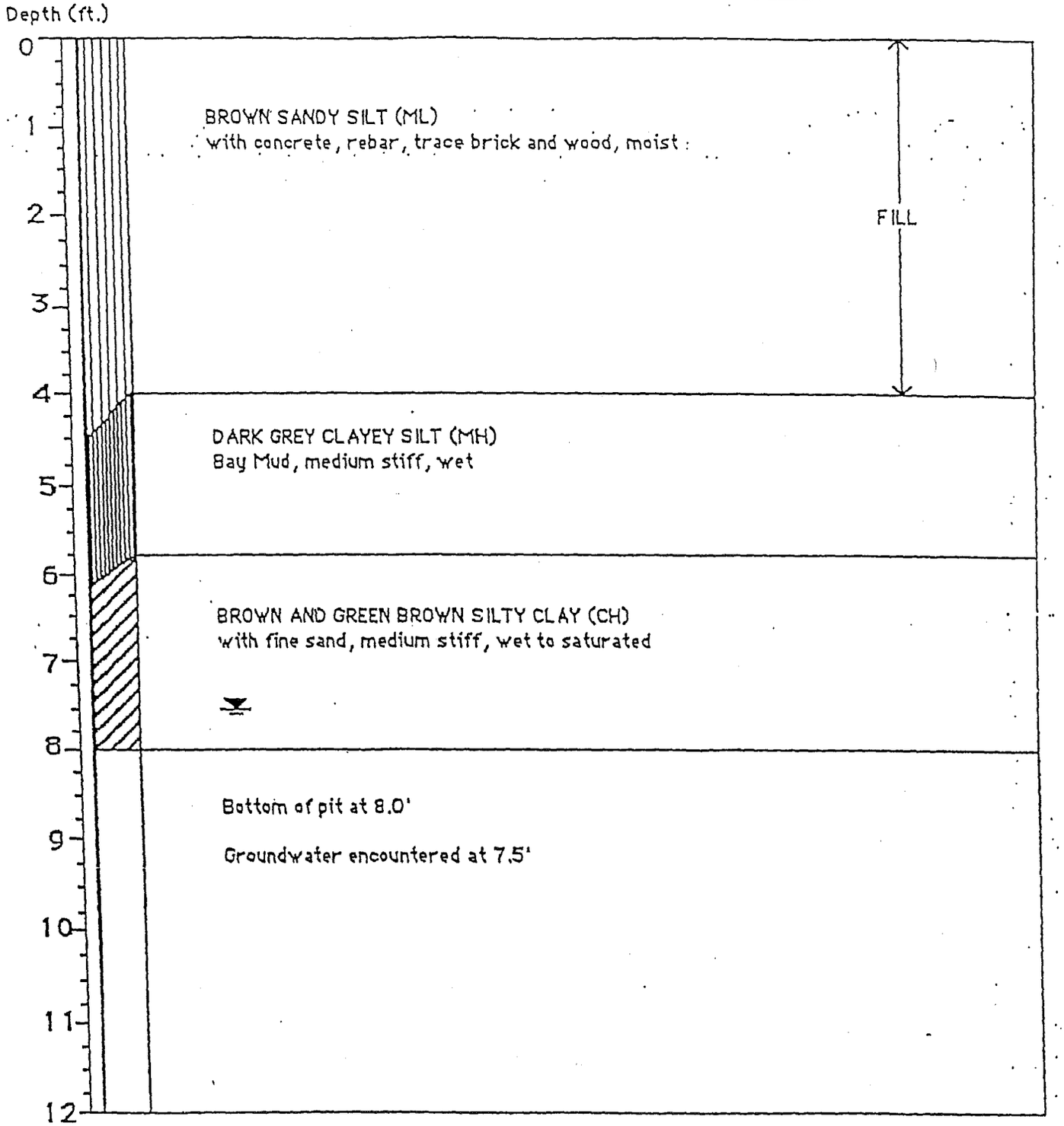


GeolResource Consultants, Inc. Geologists / Engineers / Environmental Scientists	LOG OF TEST PIT TP-A TASMAN FINAL WETLANDS MITIGATION GUADALUPE RIVER SITE SAN JOSE, CALIFORNIA	FIGURE 9
Job No. 1935-000 Appr: AMT Date 12/96		

LOG OF TEST PIT TP-B

Equipment: John Deere 690 Excavator

Elevation: ±0 Date: 11/20/96



GeolResource Consultants, Inc.
Geologists / Engineers / Environmental Scientists

Job No. 1935-000 Appr: AOT Date 12/96

LOG OF TEST PIT TP-B
TASMAN FINAL
WETLANDS MITIGATION
GUADALUPE RIVER SITE
SAN JOSE, CALIFORNIA

FIGURE 10

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS				SOIL DESCRIPTION
COARSE GRAINED SOILS over 50% coarser than #200 sieve	GRAVELS over half of coarse fraction larger than No. 4 sieve	clean gravels with little or no fines	GW	Well Graded Gravels, Gravel - Sand Mixtures
			GP	Poorly Graded Gravels, Gravel - Sand Mixtures
		gravels with over 12% fines	GM	Silty Gravels, Poorly Graded Gravel - Sand - Silt Mixtures
			GC	Clayey Gravels, Poorly Graded Gravel-Sand-Clay Mixtures
	SANDS over half of coarse fraction finer than No. 4 sieve	clean sands with little or no fines	SW	Well Graded Sands, Gravelly Sands
			SP	Poorly Graded Sands, Gravelly Sands
		sands with over 12% fines	SM	Silty Sands, Poorly Graded Sand - Silt Mixtures
			SC	Clayey Sands, Poorly Graded Sand - Clay Mixtures
FINE GRAINED SOILS over 50% is finer than #200 sieve	SILTS AND CLAYS liquid limit less than 50		ML	Silts, Very Fine Sands, Silty or Clayey Fine Sands
			CL	Low Plasticity Clays, Sandy or Silty Clays
			OL	Low Plasticity Organic Silts and Clays
	SILTS AND CLAYS liquid limit greater than 50		MH	Micaceous or Diatomaceous Silts, Volcanic Ash, Elastic Silts
			CH	High Plasticity Clays - Fat Clays
			OH	High Plasticity Organic Silts and Clays
	HIGHLY ORGANIC SOILS		Pt	Peat and Other Fibrous Organic Soils

KEY TO SAMPLES

- Modified California
- Indicates depth of sampling with no recovery
- Hydraulically pushed Shelby Tube
- Indicates depth of Standard Penetration Test and 2" sample

KEY TO TEST DATA

Shear Strength, psf
Confining Pressure or Normal Load, psf

TxUU	750	(2600)	Unconsolidated Undrained Triaxial
TxCU	540	(2600)	Consolidated Undrained Triaxial
TxCD	800	(2600)	Consolidated Drained Triaxial
DS	500	(2000)	Direct Shear
qu/2	400		Shear Strength obtained from Unconfined Compression Test

PI = Plasticity Index
C = Consolidation Test

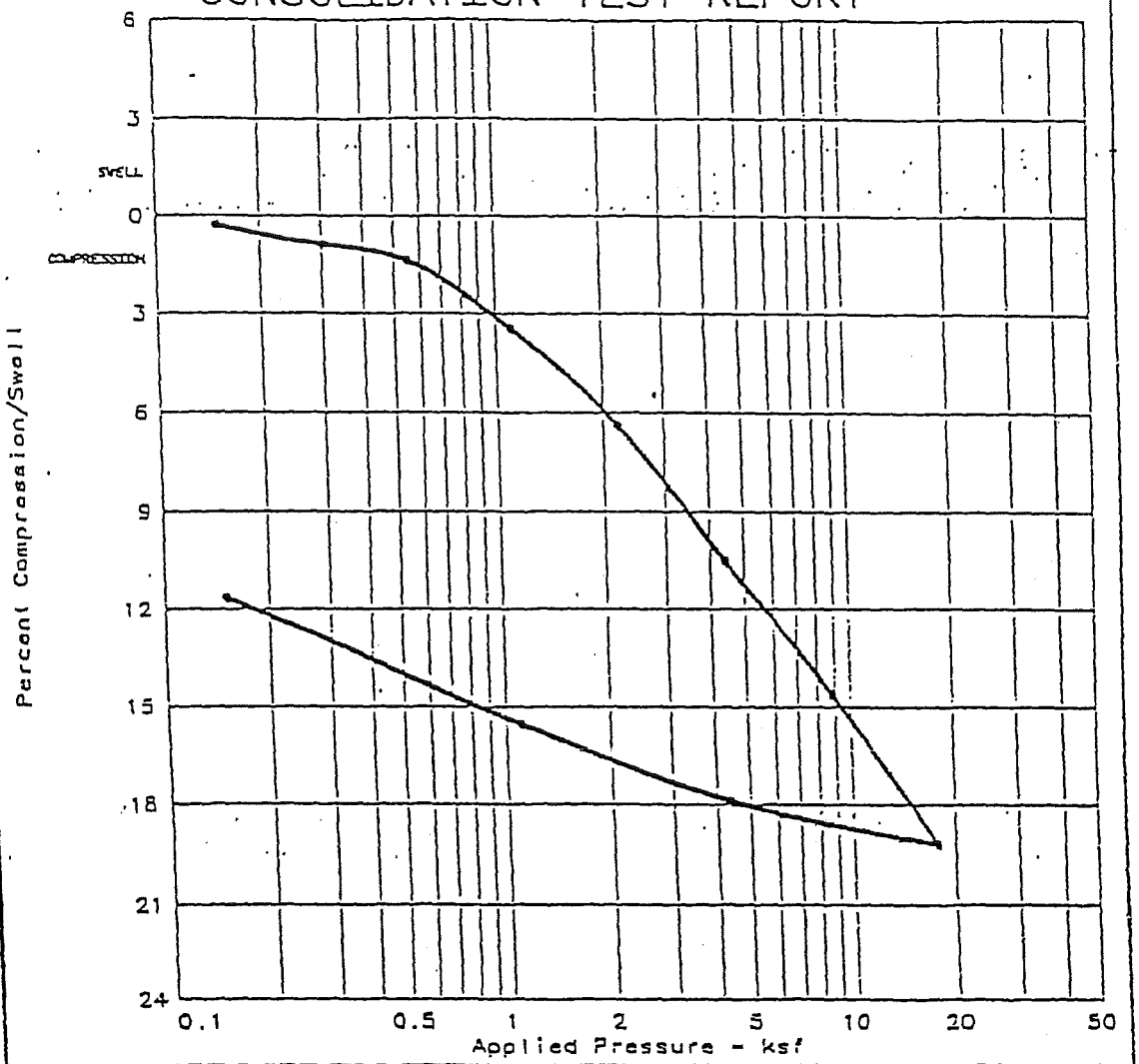
Geo/Resource Consultants, Inc.
GEOLOGISTS / ENGINEERS / ENVIRONMENTAL SCIENTISTS
505 BEACH STREET, SAN FRANCISCO, CALIFORNIA 94133

Job No. 1935-000 Appr. AJT Date 12/96

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA
TASMAN CORRIDOR PROJECT
GUADALUPE RIVER SITE
SAN JOSE, CALIFORNIA

FIGURE 11

CONSOLIDATION TEST REPORT



Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Initial void ratio
98.5 %	35.4 %	85.5			2.750	1.0075

TEST RESULTS.	MATERIAL DESCRIPTION
	gray BAY MUD
Project No.: 014-146 GRC Project: 1935 Location: B-3 @ 20' Date: 12/10/96	Remarks:
CONSOLIDATION TEST REPORT COOPER TESTING LABORATORY	Fig. No. _____

Geo/ Resource Consultants, Inc.
GEOLOGISTS / ENGINEERS / ENVIRONMENTAL SCIENTISTS
505 BEACH STREET, SAN FRANCISCO, CALIFORNIA 94133

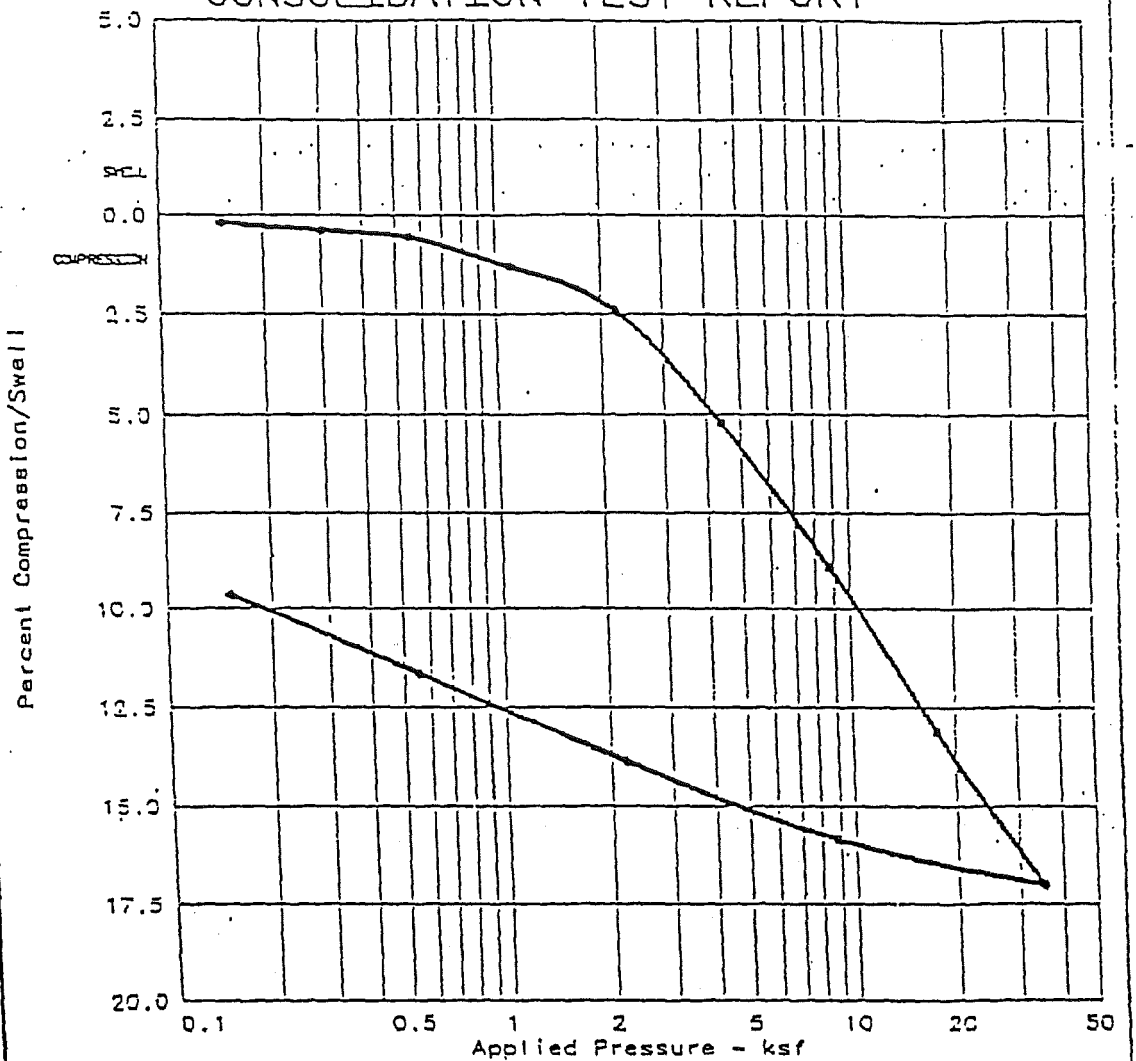
Job No. 1935-000 Appr. AOT Date 12/96

CONSOLIDATION TEST

TASMAN CORRIDOR PROJECT
GUADALUPE RIVER SITE
SAN JOSE, CALIFORNIA

FIGURE 12

CONSOLIDATION TEST REPORT



Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp.Gr.	Initial void ratio
92.2 %	23.1 %	100.6			2.700	0.5763

TEST RESULTS	MATERIAL DESCRIPTION
	yellow brown CLAY with sand
Project No.: 014-146 GRC Project: 1935 Location: B-5 @ 23.5-26.5' Date: 12/11/96	Remarks:
CONSOLIDATION TEST REPORT COOPER TESTING LABORATORY	Fig. No. ____

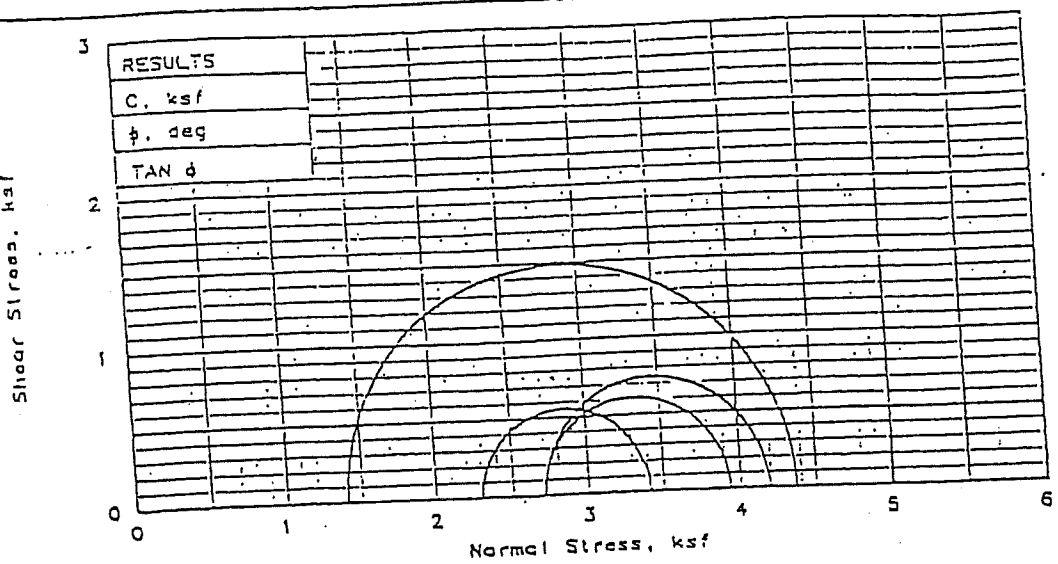
Geo/ Resource Consultants, Inc.
GEOLOGISTS / ENGINEERS / ENVIRONMENTAL SCIENTISTS
505 BEACH STREET, SAN FRANCISCO, CALIFORNIA 94133

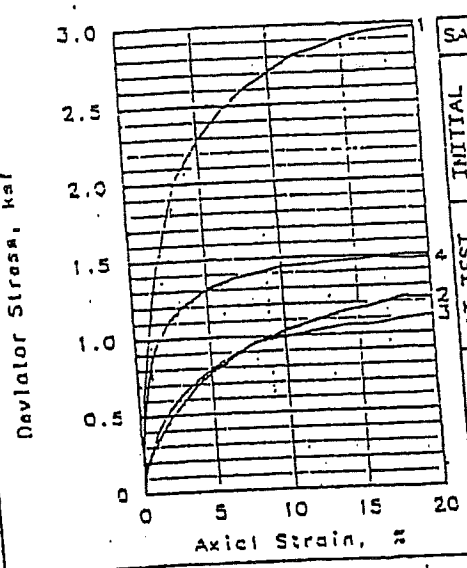
Job No. 1935-000 Appr. AOT Date 12/96

CONSOLIDATION TEST

TASMAN CORRIDOR PROJECT
GUADALUPE RIVER SITE
SAN JOSE, CALIFORNIA

FIGURE 13





SAMPLE NO.:		1	2	3	4
INITIAL	WATER CONTENT, %	19.5	34.4	44.4	24.8
	DRY DENSITY, pcf	110.2	87.5	76.9	101.8
	SATURATION, %	99.9	98.5	98.9	99.4
	VOID RATIO	0.530	0.959	1.234	0.686
	DIAMETER, in	2.870	2.400	2.410	2.870
	HEIGHT, in	6.000	5.010	5.010	6.000
AT TEST	WATER CONTENT, %	19.5	34.4	44.4	24.8
	DRY DENSITY, pcf	110.2	87.5	76.9	101.8
	SATURATION, %	99.9	98.5	98.9	99.4
	VOID RATIO	0.530	0.959	1.234	0.686
	DIAMETER, in	2.870	2.400	2.410	2.870
	HEIGHT, in	6.000	5.010	5.010	6.000
Strain rate, %/min		1.00	1.00	1.00	1.00
BACK PRESSURE, ksf		0.000	0.000	0.000	0.000
CELL PRESSURE, ksf		1.397	2.707	2.304	2.707
Devictor Stress, ksf		2.974	1.227	1.112	1.484
STRAIN, %		20.0	18.0	20.0	16.7
ULT. STRESS, ksf					
STRAIN, %					
σ_1 FAILURE, ksf		4.371	3.935	3.416	4.191
σ_3 FAILURE, ksf		1.397	2.707	2.304	2.707

TYPE OF TEST:
Unconsolidated Undrained
SAMPLE TYPE: undisturbed
DESCRIPTION: see remarks

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: 1)brown sandy CLAY
2)gray sandy CLAY w/deoris
3)dark gray organic CLAY
4)yellow brown CLAY w/sand

Fig. No.: _____

CLIENT: GRC

PROJECT: 1935

SAMPLE LOCATION: 1)B2 @ 10' 2)B3 @ 24'
3)B5 @ 19' 4)B5 @ 23.5'

PROJ. NO.: 014-146 DATE: 12/4/96

TRIAXIAL SHEAR TEST REPORT

COOPER TESTING LABORATORY

Geo/ Resource Consultants, Inc.
GEOLOGISTS / ENGINEERS / ENVIRONMENTAL SCIENTISTS
505 BEACH STREET, SAN FRANCISCO, CALIFORNIA 94133

TRIAXIAL SHEAR TEST

TASMAN CORRIDOR PROJECT
GUADALUPE RIVER SITE
SAN JOSE, CALIFORNIA

FIGUR
14

1035.000 Appr. AOT Date 12/96

Appendix B

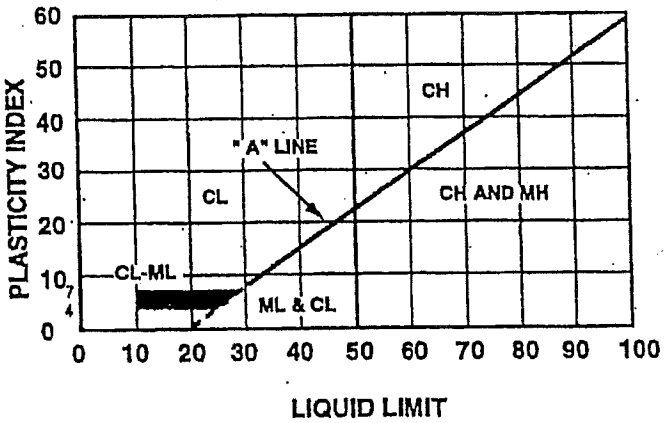
A total of seven (7) exploratory borings (EB-18, EB-19, EB-20, WB-18, WB-19, WB-20 and CB-01) were performed at the subject site under the supervision of a professional from our firm. Most borings were initially advanced by the auger method; when a depth of about 6 meters (20 feet) was achieved or if groundwater was encountered at a shallower depth, drilling mud was introduced and the remainder of the hole was completed by the rotary wash method. The diameters of the auger hole and rotary wash hole were 152 mm (6 inches) and 124 mm (4 7/8 inches), respectively. Borings were drilled during the period of September 25, 26, and 27, 2001 to depths ranging from 10.5 to 13.6 meters (34½ to 44½ feet). Samples of the soil encountered in the exploratory borings were collected with a modified California sampler 50.8 mm (2-inch) inside diameter, 63.5 mm (2½ inch) outside diameter, California sampler 63.5 mm (2.5 inches) in diameter, 82.6 mm (3.25 inch) outside diameter or a standard split spoon sampler with 35 mm (1 3/8 inch) inside diameter, 51 mm (2 inch) outside diameter. All samplers were driven into the soil with a 63.6 kg (140 pound) hammer free-falling 762 mm (30 inches). Typically, the sampler was driven 457 mm (18 inches) into the soil, and the blow count was recorded for the final 305 mm (12 inches). When very dense or hard materials were encountered and the blow count exceeded 50 blows per 152 mm (6 inches), refusal criteria was met. The amount of penetration was then recorded along with the blow count. In addition Shelby tubes were pushed hydraulically to obtain cohesive samples; the hydraulic pressure was also recorded.

After driving, the sampler was withdrawn from the borehole, the retrieved soil was examined and classified, and the brass liners containing the samples were sealed to preserve the natural moisture. The samples were taken to our geotechnical laboratory for further examination and testing.

SAMPLE CLASSIFICATION CHART

UNIFIED SOIL CLASSIFICATION SCHEME			
MAJOR DIVISIONS		SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOIL (More than 1/2 of soil >no. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > no. 4 sieve size)	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
		GP	Poorly graded gravel or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	SAND (More than 1/2 of coarse fraction < no. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOIL (More than 1/2 of soil <no. 200 sieve size)	SILTS & CLAYS Liquid Limit < 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey, fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS Liquid Limit > 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils

PLASTICITY CHART



GRAIN SIZE CLASSIFICATION		
CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL coarse (c) fine (f)	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
SAND coarse (c) medium (m) fine (f)	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.074 4.76 to 2.00 2.00 to 0.420 0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074

MOISTURE CONTENT	
DRY	-No sign of water and soil dry to touch
MOIST	-Signs of water and soil is relatively dry to touch
WET	-Signs of water and soil definitely wet to touch; granular soil exhibits some free water when densified

SOIL CONSISTENCY/RELATIVE DENSITY				
SILT, SAND AND GRAVEL	BLOWS/FT	SILT OR CLAY	UNCONFINED COMPRESSIVE STRENGTH (psf)	THUMB PENETRATION
Very loose	<4	Very Soft	< 500	Very easily - inches
Loose	5-10	Soft	500 - 1000	Easily - inches
Medium Dense	11-30	Medium (firm)	1000 - 2000	Moderate effort - inches
Dense	31-50	Stiff	2000 - 4000	Indented easily
Very Dense	>50	Very Stiff	4000 - 8000	Indented by nail
		Hard	> 8000	Difficult by nail

CLASSIFICATION MODIFIERS	
TRACE	0 - 12%
SOME	12 - 30%
± MODIFIERS	

URS

Figure B-1

Project: LOWER GUADALUPE RIVER
Location: Santa Clara County, CA

Log of Boring LEGEND

Date Drilled:
Type of Boring: (as noted)
Hammer/drop: (as noted)

Remarks:

Surface Elevation: feet (approx.)

Depth, feet	Samples	Blows/ft	Pocket Pen. (tsf)	Graphic Log	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Other Tests/ Remarks
					Arrow denotes bottom of fill layer FILL				
5					2 inch inside diameter Modified California sample				
10					2 inch outside diameter Standard Split Spoon sample (Standard Penetration Test)				
					3 inch outside diameter Shelby tube sample				
15		350 psi			Hydraulic Pressure required to push Shelby tube sampler				
20		29			Blow count with 140-lb hammer falling 30 inches for 12 inches of penetration				
25		50/ 5"			Blow count with 140-lb hammer falling 30 inches for 5 inches of penetration				
30					Groundwater level at time of drilling				
					Groundwater at a time after drilling (as specified)				
35			3.0		**KEY TO LABORATORY TESTS** PP = Pocket Penetrometer reading in tons per square foot (tsf)			-	PP = 3.0tsf
40					LL = Liquid Limit (%) PI = Plasticity Index (%) NOTE: PI = LL - (Plastic Limit [%])				LL = 42 PI = 21
					+#4 = Percentage of material retained on #4 sieve -#200 = Percentage of material passing #200 sieve				+#4 = 13% -#200 = 10%
45									
50									

Project: LOWER GUADALUPE RIVER
Location: Santa Clara County, CA

Log of Boring EB-18

Date Drilled: 9/25/01
Type of Boring: Rotary Wash
Hammer/drop: 140lb/30in

Remarks: North of Gold Street
Surface Elevation: 15.1 feet (approx.)

Depth, feet	Samples	Blows/ft	Pocket Pen. (tsf)	Graphic Log	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Other Tests/ Remarks
					6 inches Aggregate Base				
	1	34	>4.5		Lean CLAY (CL) with gravel - FILL Stiff, moist, gray brown and brown				
5	2	31			Brown, trace gravel	23	101	3740	Began rotary wash at 5'
10	3	27	3.0						
	4	54			Gravelly lean CLAY (CL) FILL Stiff, very moist, gray; with bits of concrete, wire and wood				No recovery, sample #5 immediately refused on rocks
	5				FILL				
15	6	38							No recovery
	7	21			Lean CLAY (CL) Medium, very moist, dark gray and brown mottled, trace well graded gravel				
20	8	8	1.0		Dark gray to black, with some fine sand lenses, slight organic odor, increase in plasticity				
25	9	16			Dark gray and gray brown, trace organics	24	100		
30	10	12			Poorly graded SAND (SP-SM) with silt Medium dense, wet, gray and brown mottled				
					Lean CLAY (CL) Stiff, moist, brown with gray mottled, trace sand				
35	11	25			Occasional thin gray Silty Sand (SM) lenses	23	100	2300	
40	12	16	1.5						
	13	49			Poorly graded SAND (SP-SM) with silt Dense, moist, gray, fine sand				
45					BOTTOM OF BORING AT 44-1/2 FEET				
50									

Project: LOWER GUADALUPE RIVER
Location: Santa Clara County, CA

Log of Boring EB-19

Date Drilled: 9/25/01
Type of Boring: Rotary Wash
Hammer/drop: 140lb/30in

Remarks: South of Gold Street
Surface Elevation: 14.1 feet (approx.)

Depth, feet	Samples	Blows/ft	Pocket Pen. (tsf)	Graphic Log	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Other Tests/ Remarks
					6 inches Aggregate Base ??				
	1	42	>4.5		Lean CLAY (CL) FILL Very stiff, moist, dark brown				
5	2	23	3.7		Lean CLAY (CL) with gravel - FILL Very stiff, moist, gray and brown				Began rotary wash at 5'
10	3	24							No recovery
					Gravelly lean CLAY (CL) FILL Gray and brown mottled, with concrete FILL				
	4	5			Fat CLAY (CH)	60	61	900	
15	5	5			Medium, moist, black to dark gray, some organics, organic odor				No recovery
20	6	8			With fine sand	24	95	1040	
25	7	14							No recovery
	8								
30					No sand				
35	9	7	0.5						
40	10	7	1.0		Slight organic odor				
	11	10	0.7						
45					BOTTOM OF BORING AT 44-1/2 FEET				
50									

Project: LOWER GUADALUPE RIVER
Location: Santa Clara County, CA

Log of Boring EB-20

Date Drilled: 9/26/01
Type of Boring: Rotary Wash
Hammer/drop: 140lb/30in

Remarks: South of Gold Street
Surface Elevation: 14.4 feet (approx.)

Depth, feet	Samples	Blows/ft	Pocket Pen. (tsf)	Graphic Log	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Other Tests/ Remarks
					6 inches Aggregate Base				
	1	35	>4.5		Lean CLAY (CL) FILL Very stiff, moist, brown, trace gravel				
5	2	38	3.5		Brown and dark brown				Began rotary wash at 5'
10	3	17	2.5						
	4	19	2.3		Lean CLAY (CL) with gravel - FILL Medium, gray to black, with organics	26	97		LL=42 PI=24
	5	13			FILL				
15	6	13	1.3		Lean CLAY (CL) Medium to stiff, very moist, dark gray and gray brown, trace organics				
20	7	16				30	92	2530	
					Brown with some gray brown mottling, some light brown clay nodules				
25	8	21	3.0						
30	9	12							
					Sandy lean CLAY (CL) Soft to medium, very moist, light brown				
					Lean CLAY (CL) Soft to medium, moist, brown				
35	10	15	1.0						
40	11	9	0.8		Gray				
	12	10				33	87	680	
45					BOTTOM OF BORING AT 44-1/2 FEET				
50									

Project: LOWER GUADALUPE RIVER
Location: Santa Clara County, CA

Log of Boring WB-18

Date Drilled: 9/26/01
Type of Boring: Rotary Wash
Hammer/drop: 140lb/30in

Remarks: South of Gold Street
Surface Elevation: 14.4 feet (approx.)

Depth, feet	Samples	Blows/ft	Pocket Pen. (tsf)	Graphic Log	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Other Tests/ Remarks
					6 inches Aggregate Base				
	1	33	4.0		Lean CLAY (CL) FILL Stiff, moist, brown and black				
5	2	8	2.0		Medium, gray-brown				Began rotary wash at 5'
10	3	4	1.0		FILL				
	4	6			Lean CLAY (CL) Medium, moist, gray brown with brown mottling, with roots, becomes darker and organics increase with depth	32	90	1390	
15	5	4	0.5		Sandy lean CLAY (CL) Medium, moist, gray, fine sand				
					Lean CLAY (CL) Gray brown and brown mottled				
					Wood				
20	6	15			Fat CLAY (CH) Medium, moist, dark gray, with fine roots below 21 feet, organic odor				No recovery
	7	11				34	85	1810	LL=60 PI=32
25	8	5			Light gray with brown mottling	38	83	1680	
30	9	10			Clayey SAND (SC) Loose, moist, dark gray				
35	10	4			Lean to fat CLAY (CL/CH) Medium, moist, gray, organic odor, with black organic stringers	60	63	1210	
40	11	4	0.6		With sand				
	12	16							
45					BOTTOM OF BORING AT 44-1/2 FEET				
50									

Project: LOWER GUADALUPE RIVER
Location: Santa Clara County, CA

Log of Boring WB-19

Date Drilled: 9/27/01
Type of Boring: Rotary Wash
Hammer/drop: 140lb/30in

Remarks: South of Gold Street
Surface Elevation: 18.3 feet (approx.)

Depth, feet	Samples	Blows/ft	Pocket Pen. (tsf)	Graphic Log	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Other Tests/ Remarks
					6 inches Aggregate Base				
	1	19	>4.5		Lean CLAY (CL) FILL Stiff, damp, gray-brown to brown				
5	2	19	2.5		Moist, with gravel, broken glass				
10	3	32			Medium to stiff, damp, decrease in plasticity	14	107	1710	
	4	66			Increase in plasticity				
	5	52	>4.5		With gravel				
15	6	44							
	7	44	>4.5						
	8	48							
20	9	50/5"			Clayey GRAVEL (GC) FILL Very dense, moist, brown and gray, some concrete, well graded gravel, possible bridging material FILL				
25	10	15	2.5		Lean to fat CLAY (CL/CH) Medium, dark gray				
					Lean CLAY (CL) Stiff, very moist, gray				
30	11	50 140 psi			Lean CLAY (CL) with sand Medium, moist, gray brown and brown mottled, trace fine sand Stiff				
35	12	20				20	107	2330	
40	13	7			Silty SAND (SM) Dense, moist, brown				
					Lean CLAY (CL) Soft to medium, brown, becomes stiff below 42-1/2 feet				
	14	26				23	102	3380	
45					BOTTOM OF BORING AT 44-1/2 FEET				
50									

Project: LOWER GUADALUPE RIVER
Location: Santa Clara County, CA

Log of Boring WB-20

Date Drilled: 9/27/01
Type of Boring: Rotary Wash
Hammer/drop: 140lb/30in

Remarks: North of 237
Surface Elevation: 19 feet (approx.)

Depth, feet	Samples	Blows/ft	Pocket Pen. (tsf)	Graphic Log	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Other Tests/ Remarks
					6 inches Aggregate Base				
	1	17	3.5		Lean CLAY (CL) FILL Very stiff, moist, brown and dark brown, trace well graded gravel				
5	2	28	4.5						Began rotary wash at 5'
10	3	68	>4.5						
15	4	46							
	5	45							
	6	24			Dark gray brown	19	99	4000	
20	7	30							
	8	38			With in gravel FILL				
25	9	18	2.3		Lean CLAY (CL) Stiff, moist, gray and brown mottled, with occasional gravel				
30	10	4			Sandy lean CLAY (CL) Soft, very moist, brown and gray brown mottled	23	101	610	
35	11	42	4.5		Lean CLAY (CL) Stiff to very stiff, moist, dark brown with gray mottling				
					Silty SAND (SM) Brown, trace fine gravel				
40	12	2			Lean CLAY (CL) Soft, very moist, brown, medium below 43 feet, trace fine sand				
	13	50 140 psi							
45					BOTTOM OF BORING AT 44-1/2 FEET				
50									

Project: LOWER GUADALUPE RIVER
Location: Santa Clara County, CA

Log of Boring CB-01

Date Drilled: 9/25/01
Type of Boring: Rotary Wash
Hammer/drop: 140lb/30in

Remarks: Gold Street Bridge
Surface Elevation: 4.9 feet (approx.)

Depth, feet	Samples	Blows/ft	Pocket Pen. (tsf)	Graphic Log	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Other Tests/ Remarks
					Fat CLAY (CH) FILL Soft, wet, gray to black, some organics, organic odor				
	1	2							
5	2	5				33	88	290	
	3	6			Trace glass; gray, brown and black	34	89	730	
					FILL				
	4	6	1.5		Fat CLAY (CH) Soft to medium, wet, dark brown to gray, slight organic odor				Began rotary wash at 8'
10	5	5				41	78	1050	
15									
	6	5	0.5						
20									
	7	4			Trace sand	34	83	290	
25									
					Sand				
	8	5							No recovery
30									
					Fat CLAY (CH) with sand Soft to medium, wet, olive gray, trace fine gravel, fine sand				
	9	8							
35					BOTTOM OF BORING AT 34-1/2 FEET				
40									
45									
50									

Project: LOWER GUADALUPE RIVER
Location: Santa Clara County, CA

Log of Boring CB-14

Date Drilled: 9/26/01
Type of Boring: Rotary Wash
Hammer/drop: 140lb/30in

Remarks: Gold Street Bridge
Surface Elevation: 6.2 feet (approx.)

Depth, feet	Samples	Blows/ft	Pocket Pen. (tsf)	Graphic Log	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Other Tests/ Remarks
	1	49			Clayey SAND (SC) with gravel - FILL Dense, damp, gray FILL				
	2	10	2.0		Lean CLAY (CL) with sand Stiff, moist, brown and gray brown mottled				
5	3	9				22	82		
	4	4			Fat CLAY (CH) Soft, moist, gray, organic material, organic odor, well graded sand, fine gravel	73	54	590	
	5	41				63	60	630	
10	6	41			Silty GRAVEL (GM) with sand Dense, wet, brown to gray	12	110		Began rotary wash at 10'
	7	26			Clayey GRAVEL (GC) with sand Dense, wet, gray brown, well graded sand, fine gravel Light brown Silty Sand (SM) from 12 to 13 feet				Sleeve #1 full of crushed shells
15					Lean CLAY (CL) with gravel Hard, moist, light brown				
	8	5	0.5		Fat CLAY (CH) Soft to medium, moist, dark gray, organic odor, some organics				
20									
	9	4				42	79	580	
25					Silty SAND (SM) Loose, moist, gray, fine sand				
	10	5							
30									
	11	7			Increase in plasticity of fines	23	106		+#4=0 -#200=40%
35					BOTTOM OF BORING AT 33-1/2 FEET				
40									
45									
50									

Project: LOWER GUADALUPE RIVER
Location: Santa Clara County, CA

Log of Boring CB-15

Date Drilled: 9/26/01
Type of Boring: Rotary Wash
Hammer/drop: 140lb/30in

Remarks: Gold Street Bridge Near Yacht Club
Surface Elevation: 4.5 feet (approx.)

Depth, feet	Samples	Blows/ft	Pocket Pen. (tsf)	Graphic Log	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf	Other Tests/ Remarks
					Lean CLAY (CL) with gravel - FILL Soft, moist, brown, some roots FILL				
	1	7	1.0		Fat CLAY (CH) Soft, moist, gray, trace sand				
	2	3				43	74	650	
5	3	4			∇	38	79	380	
	4	2							Began rotary wash at 6'
	5	2				63			
10	6	2	<0.5						
					Clayey SAND to Sandy fat CLAY (CH/SC) Soft to medium, wet, gray, slight organic odor, thin fine to medium sand lenses				
	7	2				51	70	330	
15									
	8	5				42	76	610	
20									
	9	10				28 41	90 76	360 1040	
25					Fat CLAY (CH) Soft to medium, wet, grayish brown and black, trace fine to medium sand, slight organic odor				
	10	6				64	60	810	
30									
	11	7			With fine sand	31	90	1330	
35					BOTTOM OF BORING AT 33-1/2 FEET				
40									
45									
50									

Appendix C

Appendix C
Laboratory Testing Program

A laboratory-testing program was implemented to obtain engineering properties from selected soil samples during the field exploration. The following tests were performed:

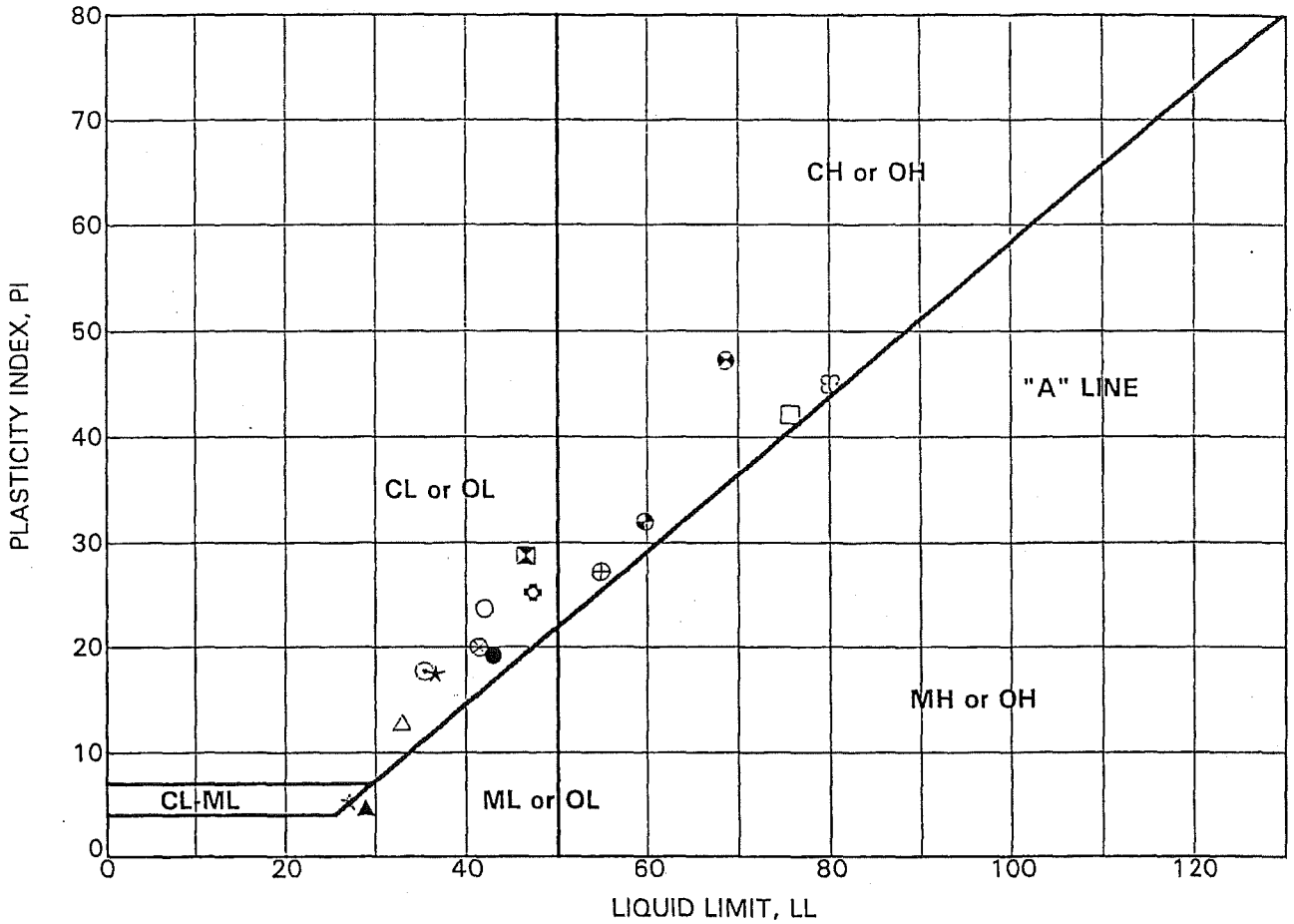
1. Moisture Content And Unit Weight: In-situ moisture content and in-situ dry unit weight was determined on selected soil samples in accordance with ASTM D2216 and D2937.

2. Unconfined Compressive Strength: The unconfined compressive strength was measured on samples retrieved from the sampler brass liners in accordance with ASTM D2166.

3. Atterberg Limits: Liquid limit (LL) and plastic limit (PL) were determined on selected samples using the procedure given in ASTM D4318. The plasticity index (PI) was computed from these values using the expression PI = LL – PL.

4. Gradation Analyses: Gradation (Sieve) analyses were performed on selected samples to determine the total percentage of gravel (retained by the No. 4 standard sieve) and fines (passing the No. 200 sieve or finer than 74 μm). The tests were performed in accordance with the standard procedure specified by ASTM D422. Wash sieves were performed on selected samples to determine the percentage of fines. Percentages of gravel and fines are also shown in Boring Logs.

Laboratory test results are shown in Figures C-1 and C-2 and other unnumbered figures are included in the Boring Logs (Appendix B).



Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
CB-05	10	28	●	38	43	24	19	Lean CLAY (CL)
CB-07	1	2	⊠	23	47	18	29	Lean CLAY (CL) with sand
CB-09	11	33	▲	23	29	24	5	SILT (ML) with sand
CB-10	4	7	★	29	37	19	18	Lean CLAY (CL)
CB-12	3	7	⊙	26	35	18	17	Lean CLAY (CL)
CB-13	9	26	✿	28	47	22	25	Lean to fat CLAY (CL/CH)
EB-20	4	12	○	26	42	18	24	Lean CLAY (CL) with gravel
EB-23	6	22	△	24	33	20	13	Sandy lean CLAY (CL)
EB-26	5	16	⊗	26	41	21	20	Lean CLAY (CL) with sand
EB-27	6	26	⊕	48	55	28	27	Fat CLAY (CH)
EB-27	8	36	□	56	76	34	42	Fat CLAY (CH) with sand
EB-28	5	21	⊖	27	69	21	48	Fat CLAY (CH) with sand
WB-18	7	22	◕	34	60	28	32	Fat CLAY (CH)
WB-23	8	36	☆	24	27	22	5	Sandy, silty CLAY (CL-ML)
WB-25	3	11	⊠	36	80	35	45	Fat CLAY (CH) with sand

Project: LOWER GUADALUPE RIVER
Project Number: 95-00112030.00

PLASTICITY CHART

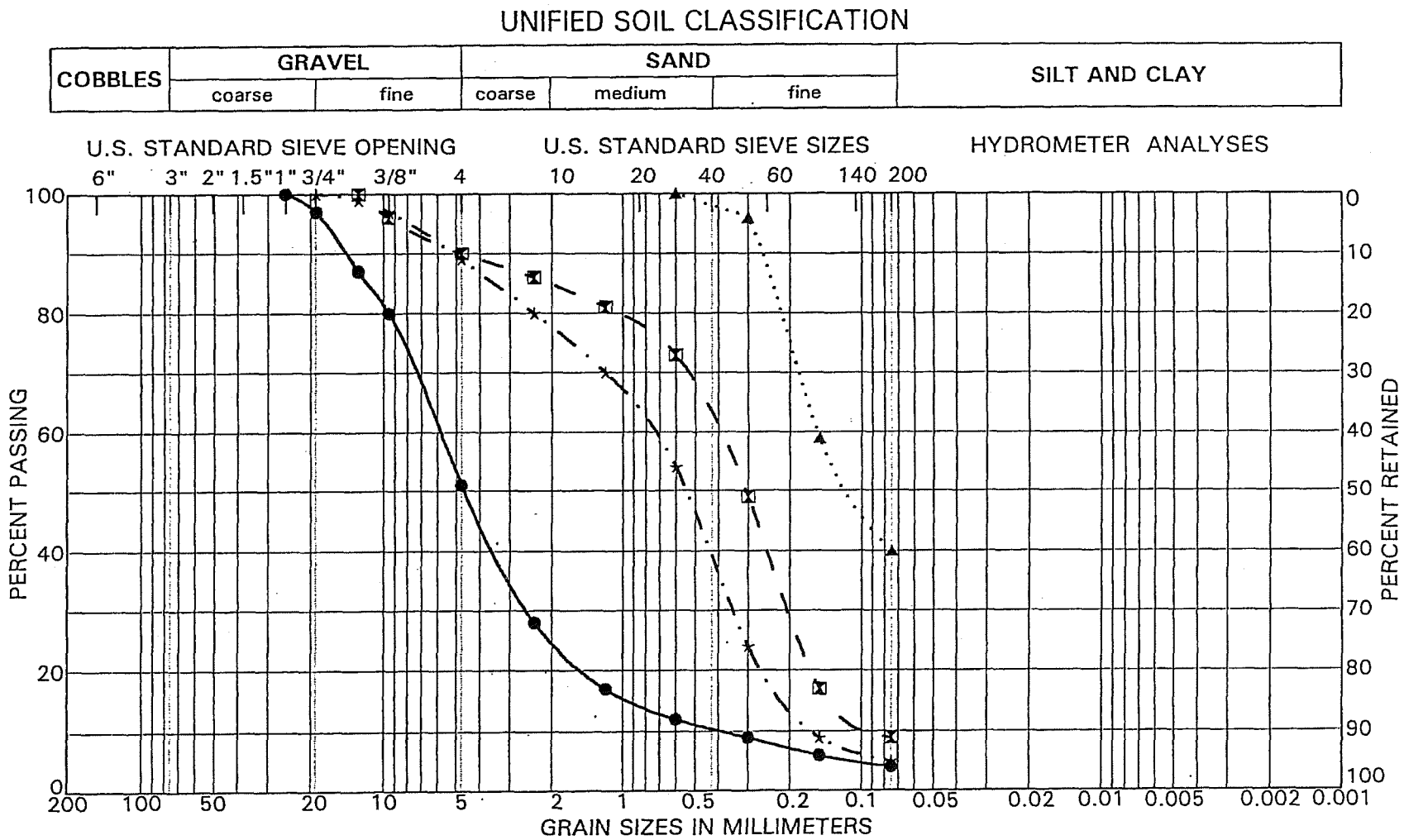
Figure C-1

11/30/01 JATT94 01030

URS

Project : LOWER GUADALUPE RIVER
Project No. 95-00112030.00

GRAIN SIZE DISTRIBUTION CURVES



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	Classification
CB-09	8	18	●			Well-graded GRAVEL (GW) with sand
CB-11	8	18	⊠			Poorly graded SAND (SP-SM) with silt
CB-14	11	33	▲			Silty SAND (SM)
EB-22	12	51	*			Poorly graded SAND (SP)

Fig. C-2

URS

11/30/01 JSIEVE 01030

Appendix D

URS Greiner Woodward Clyde
A Division of URS Corporation

April 14, 2000
95-973017NB.00

55 South Market Street, Suite 1650
San Jose, CA 95113
Tel: 408.297.9585
Fax: 408.297.6962
Offices Worldwide

Mr. Mohammad Khan
Santa Clara Valley Water District
5750 Almaden Expressway
San Jose, CA 95118-3686

Subject: Preliminary Geotechnical Parameters
Phase I Flood Protection Alternatives Analysis
Lower Guadalupe River Flood Protection Project
Santa Clara County, California

Dear Mr. Khan:

As requested by Mr. John Rogers of CH2MHill, we have developed preliminary geotechnical parameters for use by their Structural Engineer in evaluating the viability of various flood protection alternatives along the project alignment. Mr. Rogers forwarded a list of specific questions. Those questions and our responses follow.

Additionally, Mr. Rogers asked about the potential for future settlement along the alignment. Apparently, the District provided data that suggests as much as 1 foot of settlement has occurred since 1983. A brief discussion of settlement also is included.

RESPONSES TO CH2MHILL QUESTIONS

What are the lateral soil pressure parameters (gamma & phi)?

Estimated values of gamma and phi are summarized in Table 1, corresponding to the general soil conditions we encountered throughout the various project reaches.

What is the soil bearing capacity, say 5' from the top of the levees?

Estimates of bearing capacity within the existing levee fill are provided in Table 1 for dead plus live load, and for all loads including seismic. As above, the bearing pressure values correspond to the conditions encountered within the corresponding alignment reaches.

What is the bearing capacity at the toe of the levee?

Table 1 also provides preliminary estimates of the bearing capacity at the inboard toe of the levees.

What type of piling would be feasible to drive through the levee (concrete, steel H-section or pipe pile)?

In recent years, square precast prestressed concrete piles generally have been the most economical piles to drive in the Bay Area. Typically, 12-inch and 14-inch sizes have been used to support multi-story buildings and highway bridges. Based on the compositions of the levees (i.e., predominantly clay soils), we anticipate it would be feasible to install precast prestressed concrete piles through the levees and into the underlying native alluvial soils. It is conceivable some

localized obstructions could be encountered. However, predrilling could be used to minimize potential problems during driving.

Steel H-pile and pipe pile alternatives also would be feasible, but could be more costly to install. Another potential pile alternative would be cast-in-place concrete piers, also referred to as cast-in-drilled-hole piles (CIDH). We would envision the minimum diameter of CIDH piles might be on the order of 16 to 18 inches. Larger sizes (e.g., 30 to 36 inches) could be used to minimize potential caving problems associated with drilling below groundwater. Casing or the use of drilling mud also could be used to facilitate CIDH pile construction below groundwater.

What would be the length of piles for 45 tons of bearing capacity?

The conditions encountered along most of the project alignment suggest piles would derive capacity through skin friction rather than end bearing. Nonetheless, a dead plus live load design capacity of 45 tons could be achieved with pile embedment lengths in the range of about 40 to 50 feet for piles with a minimum diameter / width of about 12 to 14 inches. For preliminary purposes, we suggest a dead plus live load unit skin friction value of 500 psf be assumed when estimating the embedment length that might be required for either driven piles or CIDH piles. This value can be assumed for the levee fill and underlying native alluvium. A one-third increase can be applied for seismic loading conditions.

What is the seismic rock acceleration?

Based on a seismic review of the project vicinity, we expect the controlling seismic source to be a Mw 8 earthquake along the 1906 segment of the San Andreas fault. This event has the potential of generating median peak horizontal ground accelerations of 0.31 to 0.33 g. However, please refer to Section 2, Site Geology and Seismicity, of our July 14, 1999 draft geotechnical investigation report for a more comprehensive discussion of the seismicity.

What is the depth to bedrock or rock-like materials?

The depth to bedrock is estimated to be in excess of 93 m (300 ft) for the general project area. Please refer to Section 2, Site Geology and Seismicity, of our July 14, 1999 draft geotechnical investigation report.

ESTIMATED FUTURE LEVEE SETTLEMENT

For preliminary purposes, we have assumed that future raising of the levees would be limited to approximately 0.9 meter (3 feet) for all seven levee sections discussed in our draft geotechnical report. Our estimates are presented below:

Section	Incremental Estimated Future Settlement Due to 0.9 meter (3 feet) of New Fill	
	mm	inches
1	203	8
2	171	6¾
3	267	10½

4	38	1½
5	235	9¼
6	6	¼
7	19	¾

Therefore the future estimated settlement due to 0.9 meter (3 feet) of new fill placed on the levees generally north of Montague Expressway will be of the order of 254 mm (10 inches) and south of Montague Expressway will be of the order 19 mm (3/4 inch). However the transition zone between these two areas is gradual and could be of the order of 1,000 meters or more. The reasons for the wide variation in estimated settlement are: (1) highly compressible clay such as Bay mud or fat clay is present in the Alviso area, (2) some clays are overconsolidated by a ratio of 2 or 3 or greater, and (3) thickness of clay layers is highly variable along the project alignment.

LIMITATIONS

The geotechnical opinions and parameters presented herein are preliminary and intended to assist CH2MHill in the evaluation of feasible flood protection alternatives. They have been developed with the standard of care of the geotechnical engineering profession. No other warranties are included, either express or implied, as to the professional advice presented. More detailed geotechnical analysis should be conducted as part of the final design.

If any questions should arise, please contact our office.

Sincerely,

Paul J. Boddie, G.E.
Geotechnical Department Manager

Attachment

Cc: Mr. John Rogers, CH2MHill

Table 1
Summary of Preliminary Geotechnical Design Parameters
Lower Guadalupe River Flood Control Improvement Project
Santa Clara County, California

Reach	Moist Unit Weight (pcf)	Internal Friction Angle[1] (ϕ')	Bearing Capacity at 5 feet below Crest of Levee (psf)		Bearing Capacity at Toe of Levee (psf)		Design Unit Skin Friction for Piles (psf)
			DL+LL	All Loads	DL+LL	All Loads	
A	120	29	3000	4500	2500	4000	500
B	120	29	3000	4500	2500	4000	500
C	128	28	3000	4500	2000	3000	500
D	125	30	2500	4000	1500	2500	500
E	125	30	2000	3000	1500	2500	500
F	130	31	3000	4500	1500	2500	500
G	130	28	2000	3000	1000	1500	500

Notes:
(1) Drained, i.e. effective strength parameter.

URS

October 23, 2001
95-00112030.00

Mr. David Von Rueden
CH2MHILL
1737 North First Street, Suite 300
San Jose, California 95112

Subject: Supplementary Geotechnical Parameters
Floodwall Design
Lower Guadalupe River
Santa Clara County, California

Dear Mr. Von Rueden:

As requested by Mr. Majdi Kanaan of CH2MHILL in the attached e-mail dated October 16, 2001, we have developed supplemental geotechnical parameters for the design of the levee floodwalls. Mr. Kanaan forwarded a list of specific questions. Those questions and our responses follow. Previously, we provided preliminary geotechnical parameters for levee floodwalls in a letter dated April 14, 2000 to the Santa Clara Valley Water District (Project Number 95-973017NB.00).

RESPONSES TO CH2MHILL QUESTIONS

What are the earth pressure coefficients and wall friction values to be used for design of the CIDH piles?

The design geotechnical parameters for the design of the CIDH piles are presented in the table below for reaches F and G, as defined by CH2MHILL. The values of unit weight and friction angle are consistent with those provided in our letter dated April 14, 2000. The active and passive earth pressure coefficients provided herein do not include the effect of the wall friction. A schematic of the lateral pressure distribution is presented in Figure 1.

	Total Unit Weight (pcf)	Submerged Unit Weight (pcf)	Friction (1) Angle (degrees)	Wall Friction Angle	Isolation Factor	K_A	K_P
Reach F	130	67.6	31	20	1.0	0.32	2.0
Reach G	130	67.6	28	18	1.0	0.36	1.8

(1) Drained, i.e. effective strength parameter.

What is the uplift on an L-shaped spread footing foundation?

Assuming a 7.0-foot spread footing width and maximum height of water above the levee of approximately 3.0 feet, the design uplift pressure has a trapezoidal distribution, with

URS Corporation
55 South Market Street, Suite 1650
San Jose, CA 95113
Tel: 408.297.9585
Fax: 408.297.6962

maximum and minimum values of 440 pounds per square foot (psf) and 125 psf, respectively. The uplift pressure distribution is presented in Figure 2.

LIMITATIONS

The geotechnical opinions and parameters presented herein are intended to assist CH2MHILL in the design of floodwall foundations for the Lower Guadalupe River Flood Control Project. They have been developed with the standard of care of the geotechnical engineering profession. No other warranties are included, either express or implied, as to the professional advice presented.

If any questions should arise, please contact our office.

Sincerely,

Paul J. Boddie, G.E.
Geotechnical Project Manager

Attachments

cc: Mr. Majdi Kanaan, CH2MHILL

URS

Page 1 of 1

Job LOWER GUADALUPE RIVER

Project No.

Sheet ___ of ___

Description LATERAL PRESSURE DESIGN PARAMETERS

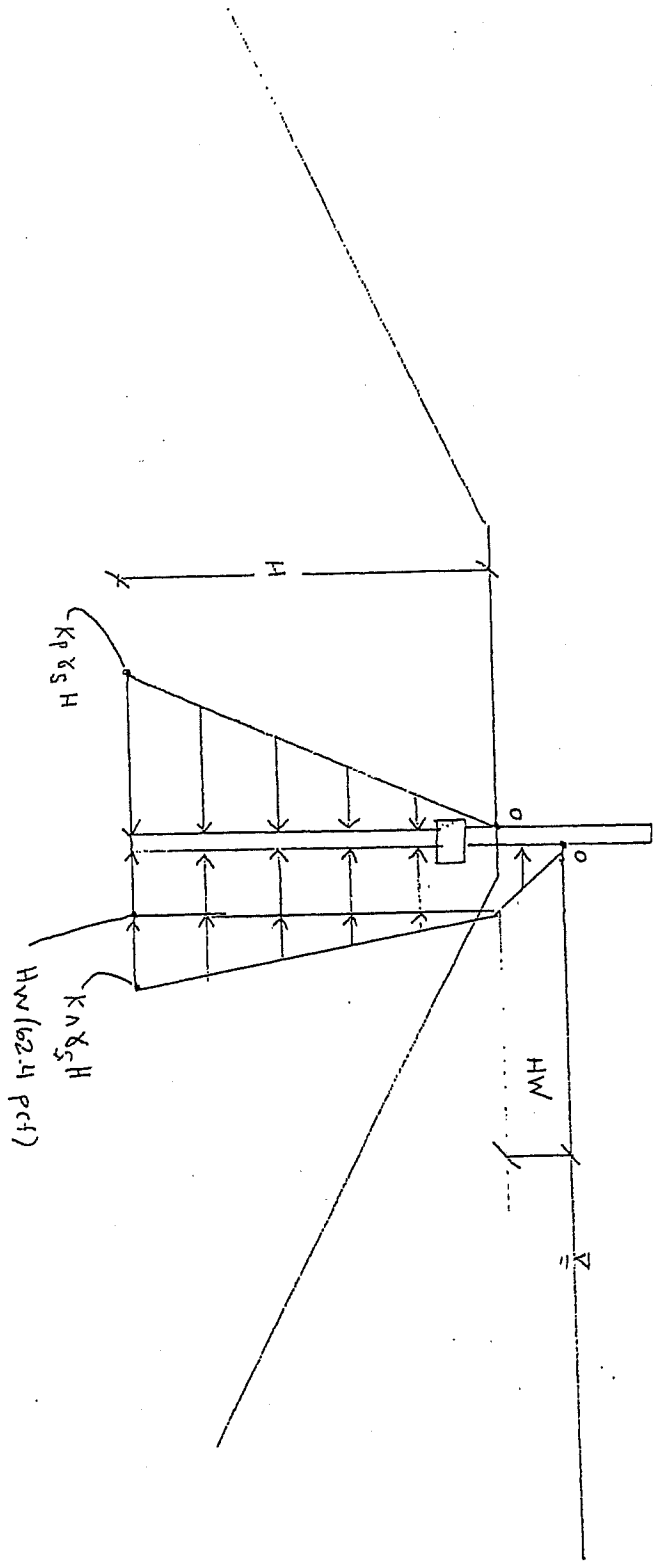
Computed by J. WERLE

Date 10/19/01

Checked by

Date

Reference



γ_s = submerged unit weight = 67.6 pcf.

Figure 1

URS

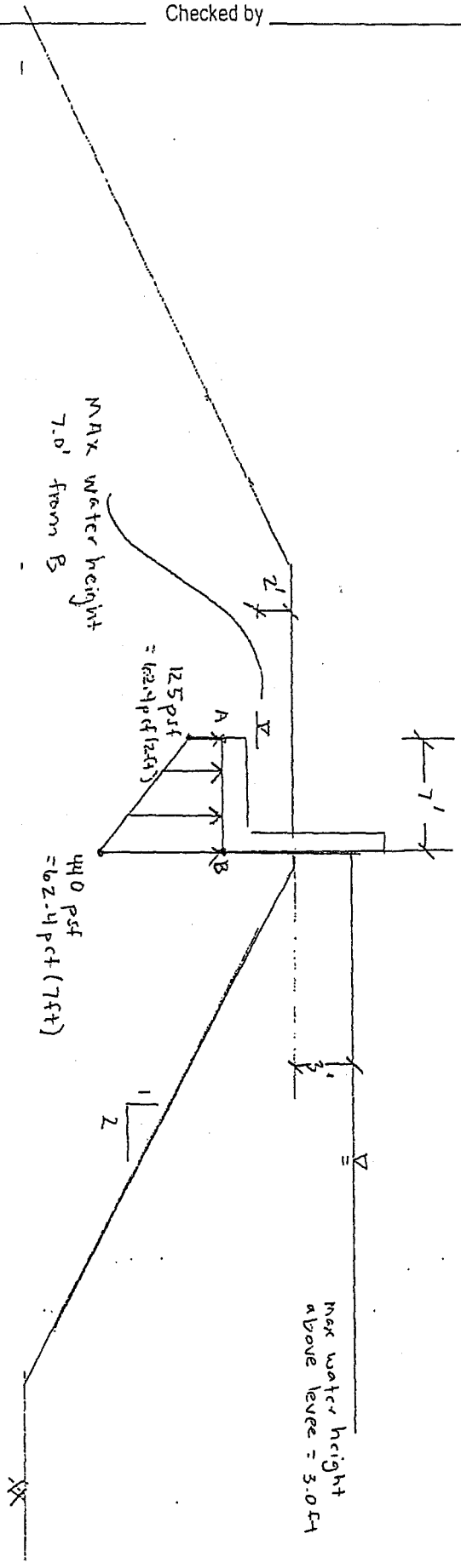
Page 1 of 1

Job LOWER GUADALUPE RIVER | Project No. | Sheet ___ of ___

Description UPLIFT on 7' shallow foundation | Computed by J. Werle | Date 10/19/01

Checked by | Date

Reference



SCALE 1" = 10'

Figure 2

"Kanaan, Majdi/SAC" <mkanaan@CH2M.com>

10/16/01 11:36 AM

To: Michael_larson@urscorp.com
cc: stephen_huang@urscorp.com, paul_boddie@urscorp.com, "Von Rueden, Dave/SJC" <DRueden@CH2M.com>, "Tamannaie, Nader/SAC" <NTamanna@CH2M.com>, "Gomez, Erika/SJC" <egomez@CH2M.com>
Subject: Lower Guadalupe River project

<<29169.tif>>
As a follow up on our conversation today, I need your recommendations at each reach and at Santa Clara mitigation levy, for the active and passive coefficients (sloping backfill) Ka , Kp as well as the wall friction angle values. Since the flood walls are located on top of the levy, the hinge point is not far enough from the flood wall to assume a flat bedding surface per Caltrans memo 22-1. To assist you, the attached Caltrans memo 22-1 shows the geometry of the wall foundation we are using assuming 16" or 24" CIDH piles. The pile cap extends 3" on each side of the pile and is 2' deep. The max wall heights are expected to be 5' or so above the levy.

We are also investigating using L -Shaped spread footing alternative. For an 8.0' high flood wall including 2.5' cover and 1.5 thick footing and under hydrostatic pressure only, the footing width we are coming up with is 7' wide. What can we assume for uplift and seepage under this scenario?

Thank you for your assistance.

Majdi kanaan/Sac
916-920-0212 Ext 361

29169.tif

IV. Foundation Design

The allowable vertical soil bearing capacity, the soil properties to be used in calculating the lateral soil bearing values and other pertinent foundation data will be shown in the foundation report that is provided by the Engineering Geology and Technical Services Branch of the Transportation Laboratory.

The passive soil pressures *shall be* increased by a factor of 1.5 for the design of laterally loaded piles located in cohesionless soils and in level ground. The increased pressure is defined as the 'EFFECTIVE' passive pressure and the increase factor of 1.5 is defined as the 'ISOLATION' factor. The 'ISOLATION' factor is a means to account for the assumption that a laterally loaded pile is resisted by a section of earth that is wider than that of the pile. A level ground condition is defined as one in which the ground surface is approximately level or , when sloping down and away from the pile, is not steeper than 10:1 for $\phi = 35°$ or 14:1 for $\phi = 25°$. A level ground condition may also be assumed when the hinge point of any adjacent down or negative slope that is steeper than 10:1 for $\phi = 35°$ or 14:1 for $\phi = 25°$ is not located closer than 2 times the pile embedment. The 10:1 and 14:1 negative slopes are approximately equal to a β/ϕ ratio of - 0.15, where β is the angle of the slope in degrees and ϕ is the angle of shearing resistance in degrees. There shall be no increase in the 'EFFECTIVE' passive pressure for piles located in cohesive soils or in sloping ground. Figure 5 illustrates the parameter for the level ground condition.

With the exception of the factors of safety for the stability of spread footings, the foundation requirements for embedment, width, depth and strength shall be determined by the Load Factor Design (LFD) Method using the loading combinations, the load factors and the strength reduction factors shown in Section II: Load Combinations.

Piles located on slopes *are to* be protected by a berm. The berm should have 305 mm minimum width and provide 152 mm minimum depth of cover above the top of pile or pile cap.

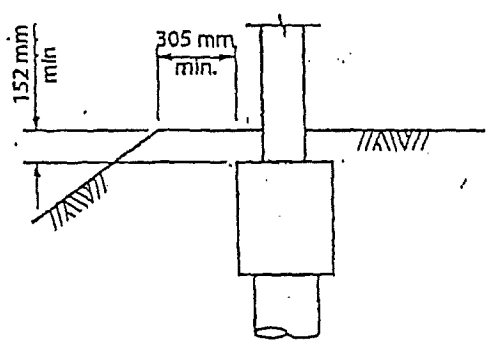


Figure 4

Appendix E

October 30, 2002
<job02-029B\Gold St to Hwy237 Soil Report1 .doc>

Mr. Stephen Huang
URS Corporation
55 South Market St., Suite 1650
San Francisco, CA 95113

SUBJECT: **Corrosion Engineering Services for the SCVWD Lower Guadalupe Flood Protection Project – Gold Street – Highway 237**

Dear Mr. Huang:

V & A Consulting Engineers (V & A) was retained by URS Corporation (URS) to perform a soil investigation as part of the Lower Guadalupe River Flood Protection Project. The objective of the investigation was to perform field-soil resistivity testing and chemical analysis of selected soil samples, and evaluate these results with respect to the possible levels of corrosion of buried concrete foundation materials, buried metallic structures, corrugated metal pipe, and prestressed concrete piles.

TEST METHODS

In attempting to predict corrosion problems associated with a particular type of buried metallic or concrete structures prior to installation, it is necessary to investigate the soil conditions the structure will encounter. Since corrosion of metal is fundamentally an electrochemical process accompanied by current flow, the electrochemical characteristics of a soil are of primary importance when evaluating corrosivity. Test methods utilized during this investigation are in accordance with Caltrans Standards and reflect the most practical methods of evaluating corrosivity. Boring samples were not available on the north end of this section. No soil chemical analysis was performed for this section.

Soil Resistivity

Resistivity of the soil was measured by V & A at 10 locations at the project site on April 3, 2002. Figure 1 shows the vicinity and project location maps. Figure 2 shows an aerial view of the project site with the soil resistivity testing locations highlighted in blue.

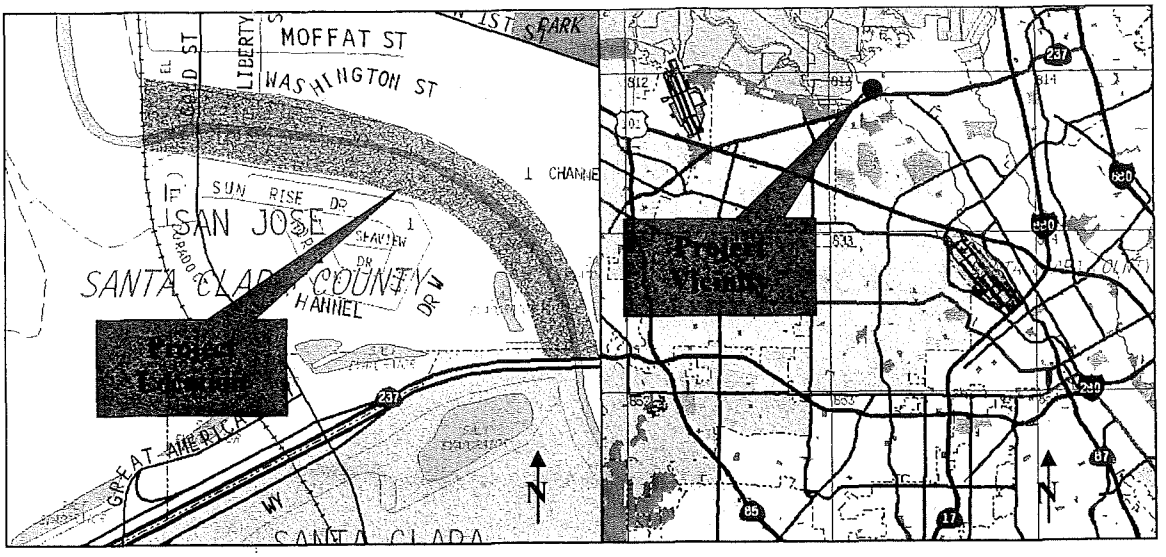


Figure 1. Vicinity and Project Site Maps for Gold Street – Hwy 237 Section

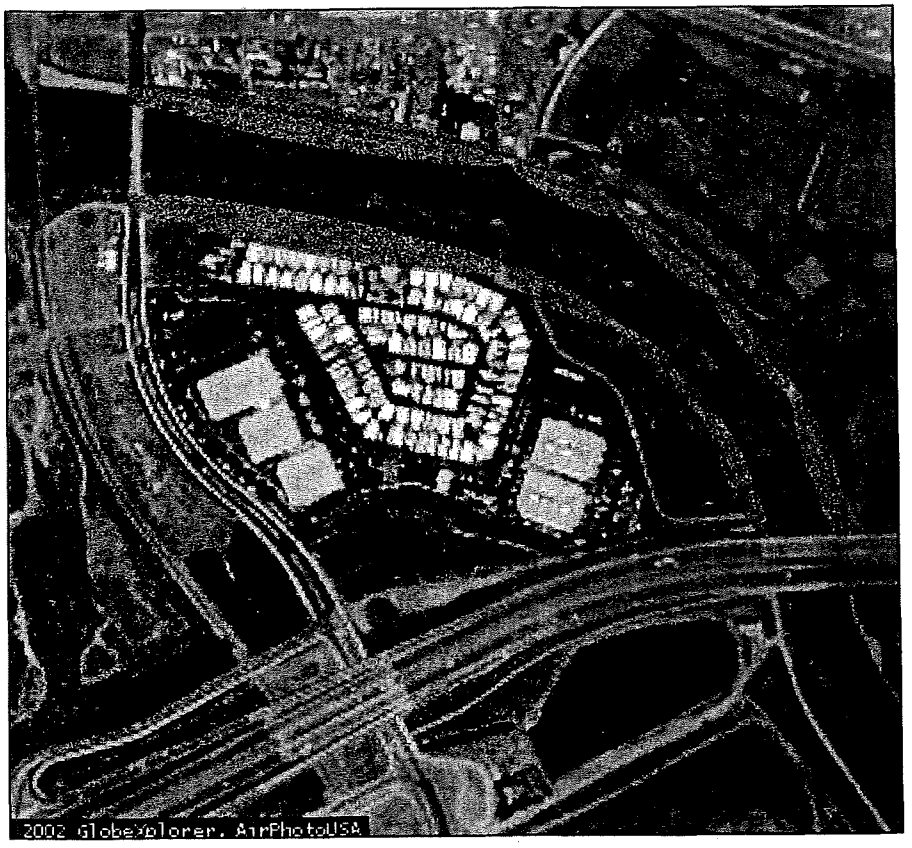


Figure2. Aerial Photograph of Soil Resistivity Testing Locations

Soil resistivity measurements were conducted by the Wenner 4-pin method, utilizing a Soil Resistance Meter, Model 400, manufactured by Nilsson Electrical Laboratory, Inc. The Wenner method involves the use of four metal probes or electrodes, driven into the ground along a straight line, equidistant from each other, as shown in Figure 3.

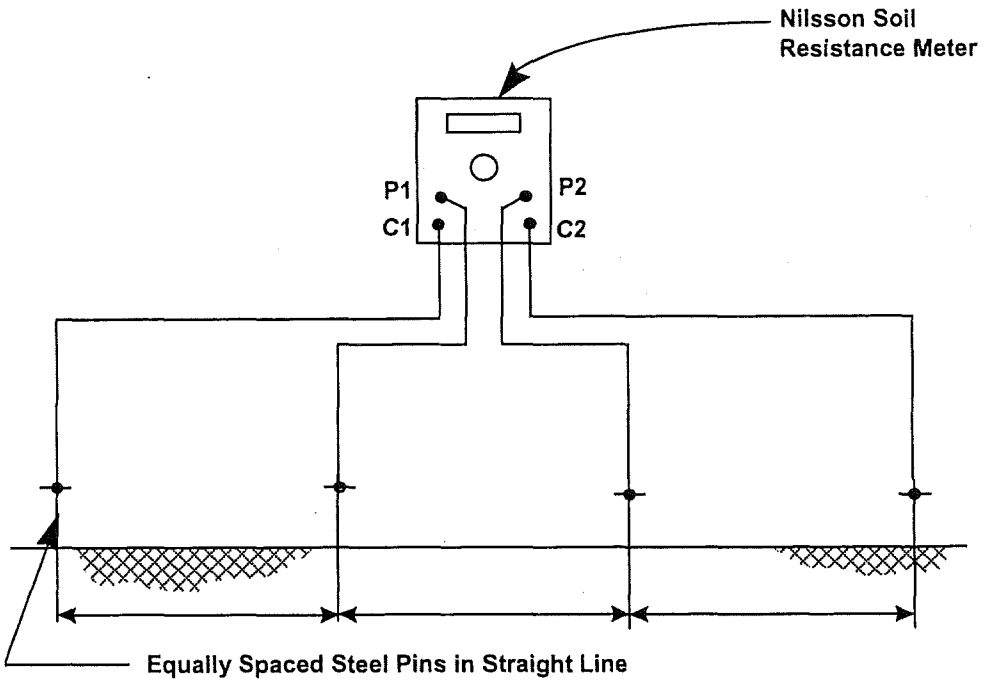


Figure 3. Wenner 4-Pin Method for Measurement of Soil Resistivity

An alternating current from the Soil Resistance Meter causes a current to flow through the soil between the outside probes, C1 and C2. Due to the resistance of the soil, the current creates a voltage gradient, which is proportional to the average resistance of the soil mass to a depth equal to the distance between probes. The voltage drop is then measured across pins, P1 and P2. Resistivity of the soil is then computed from the instrument reading according to the following formula:

$$p = 2\pi AR$$

where:

p = soil resistivity (ohm - cm)

A = distance between probes (cm)

R = soil resistance, instrument reading (ohms)

$\pi = 3.14159$

Soil resistivity measurements were conducted at probe spacings of 5, 10, 15 and 20 feet. The resistivity values obtained and listed in Table 1 represent the average resistivity of the soil to a depth equal to the pin spacing and the resistivity values for the designated layer of soil.

TEST RESULTS

Data obtained during the field resistivity testing has been summarized in tabular form for analysis and presentation. Table 1 lists the results of the field soil resistivity measurements conducted at the sites. Table 2 summarizes the results of the field soil resistivity measurements. Soil chemical analyses from borings at Highway 237 Bridge are shown in Table 3.

Laboratory Analysis of Soil Samples

Laboratory analysis was performed on soil samples provided to V & A. The testing included laboratory resistivity in the "as-received" and minimum (or saturated) condition. The minimum or saturated resistivity provides information about the soil during wet weather conditions. With the addition of distilled water to each sample, the minimum resistivity will decrease from the original "as-received" soil resistivity, in almost every instance. This decrease in soil resistivity, when saturated, indicates a possible influence from additional chemical constituents. This change can be attributed to the following factors:

1. The presence of chemicals in the "as-received" soil going into solution and providing a less resistive path for current flow.

2. Large concentrations of bicarbonate ions which tend to lower the soil resistivity but are not considered corrosive to buried steel and ductile iron structures.

Table 1. Field Soil Resistivity Test Data

Site No.	Depth (meters)	R1 (ohm) Meter Reading	R1 (ohm) Mult	"K" Factor	Resistivity (ohm-cm)	1/R1 (mhos)	Del (1/R1) Del mhos	R2 (ohm)	"K" Factor (cm)	Layer Resistivity (ohm-cm)	Depth (meters)
1	1.5	3.0	1	958	2,874	0.3			0	2,874	0 - 1.5
	3	6.0	0.1	1,915	1,149	1.7	1.3	0.8	958	719	1.5 - 3
	4.6	3.8	0.1	2,837	1,078	2.6	1.0	1.0	958	993	3 - 4.6
	6.1	3.1	0.1	3,832	1,188	3.2	0.6	1.7	958	1,612	4.6 - 6.1
2	1.5	9.6	0.1	958	920	1.0			0	920	0 - 1.5
	3	4.2	0.1	1,915	804	2.4	1.3	0.7	958	715	1.5 - 3
	4.6	2.0	0.1	2,837	567	5.0	2.6	0.4	958	366	3 - 4.6
	6.1	1.2	0.1	3,832	460	8.3	3.3	0.3	958	287	4.6 - 6.1
3	1.5	1.3	1	958	1,245	0.8			0	1,245	0 - 1.5
	3	3.9	0.1	1,915	747	2.6	1.8	0.6	958	534	1.5 - 3
	4.6	2.1	0.1	2,837	596	4.8	2.2	0.5	958	436	3 - 4.6
	6.1	1.2	0.1	3,832	460	8.3	3.6	0.3	958	268	4.6 - 6.1
4	1.5	1.0	1	958	958	1.0			0	958	0 - 1.5
	3	2.0	0.1	1,915	383	5.0	4.0	0.25	958	240	1.5 - 3
	4.6	1.0	0.1	2,837	284	10.0	5.0	0.20	958	192	3 - 4.6
	6.1	9.4	0.01	3,832	360	10.6	0.6	1.57	958	1,501	4.6 - 6.1
5	1.5	6.7	0.1	958	642	1.5			0	642	0 - 1.5
	3	1.5	0.1	1,915	287	6.7	5.2	0.2	958	185	1.5 - 3
	4.6	7.5	0.01	2,837	213	13.3	6.7	0.2	958	144	3 - 4.6
	6.1	1.2	0.1	3,832	460	8.3	5.0	0.2	958	192	4.6 - 6.1
6	1.5	2.7	1	958	2,587	0.4			0	2,587	0 - 1.5
	3	5.7	0.1	1,915	1,092	1.8	1.4	0.7	958	692	1.5 - 3
	4.6	3.1	0.1	2,837	879	3.2	1.5	0.7	958	651	3 - 4.6
	6.1	2.6	0.1	3,832	996	3.8	0.6	1.6	958	1,544	4.6 - 6.1
7	1.5	7.9	0.1	958	757	1.3			0	757	0 - 1.5
	3	5.0	0.1	1,915	958	2.0	0.7	1.4	958	1,305	1.5 - 3
	4.6	4.6	0.1	2,837	1,305	2.2	0.2	5.7	958	5,508	3 - 4.6
	6.1	2.5	0.1	3,832	958	4.0	1.8	0.5	958	525	4.6 - 6.1
8	1.5	1.2	1	958	1,150	0.8			0	1,150	0 - 1.5
	3	2.3	0.1	1,915	440	4.3	3.5	0.3	958	273	1.5 - 3
	4.6	1.0	0.1	2,837	284	10.0	5.7	0.2	958	169	3 - 4.6
	6.1	8.7	0.01	3,832	333	11.5	1.5	0.7	958	641	4.6 - 6.1
9	1.5	1.2	1	958	1,150	0.8			0	1,150	0 - 1.5
	3	3.7	0.1	1,915	709	2.7	1.9	0.53	958	512	1.5 - 3
	4.6	2.5	0.1	2,837	709	4.0	1.3	0.77	958	738	3 - 4.6
	6.1	1.2	0.1	3,832	460	8.3	4.3	0.23	958	221	4.6 - 6.1
10	1.5	7.8	0.1	958	747	1.3			0	747	0 - 1.5
	3	2.1	0.1	1,915	402	4.8	3.5	0.3	958	275	1.5 - 3
	4.6	1.9	0.1	2,837	539	5.3	0.5	2.0	958	1,911	3 - 4.6
	6.1	1.1	0.1	3,832	422	9.1	3.8	0.3	958	250	4.6 - 6.1

Table 2. Summary of Field Resistivity Results

Depth	Minimum	Maximum	Average
At Depth Resistivity			
1.5 m.	642	2,874	1,303
3.0 m.	287	1,149	967
4.6 m.	213	1,305	645
6.1 m.	333	1,188	626
Layer Resistivity			
0.0 – 1.5 m.	642	2,874	1,303
1.5 – 3.0 m.	185	1,305	545
3.0 – 4.6 m.	144	5,508	1,111
4.6 – 6.1 m.	192	1,612	758

Table 3. Soil Resistivity and Chemical Data

Item No.	Boring No.	Sample No.	Depth (meters)	Soil Resistivity		Chemical Data			
				As-Received (ohm-cm)	Minimum (ohm-cm)	pH	Sulfate (ppm)	Chloride (ppm)	Bicarbonate (ppm)
1	EB-21	4-3	3.3	64,000	1,000	7.44	2,200	22	1,600
2	CB-03	10-3	10	3,100	550	7.45	82	1,000	730
3	EB-21	8-3	7.9	5,300	1,500	7.81	46	330	5,400
4	B-5	5-3	6.1	10,100	3,100	7.68	28	79	1,900
5	EB-22	6-3	6.4	7,500	450	7.61	1,000	1,600	2,400
6	B-8	5-3	6.1	31,000	3,900	-	-	-	-
7	CB-04	6-3	4.6	6,500	1,600	-	-	-	-
8	EB-22	3-3	3.3	54,000	2,000	-	-	-	-

Since corrosion is a natural electrochemical process accompanied by the flow of electrical current, it is important to understand how easily current will travel through a medium surrounding a metal object. Resistivity is a measure of the ability of a soil to conduct an electric current. The higher the resistivity the more difficult it is for the soil to conduct a current. Soil resistivity is primarily dependent on the chemical and moisture content of the soil. The higher the level of chemical constituents the lower the soil resistivity. Additional moisture generally decreases the soil resistivity up to the point where the maximum solubility for the dissolved ionic chemicals is achieved. Beyond this point, an increase in moisture generally increases the resistivity as the chemicals become more and more diluted. Since corrosion rate depends on current flow to and from a metal and the adjacent medium, the corrosion activity of steel reinforcement in soil normally increases as soil resistivity decreases. The following table correlates resistivity values with degree of corrosivity. The interpretation of corrosivity varies among corrosion engineers. However, this table is a generally accepted guide.

Soil Resistivity (ohm-cm)	Degree of Corrosivity
0 - 500	Very Corrosive
500 - 1,000	Corrosive
1,000 - 2,000	Moderately Corrosive
2,000 - 10,000	Mildly Corrosive
Above 10,000	Negligible

Reference: NACE Corrosion Basics, page 191

Results of the soil laboratory analysis are shown in Table 3. A wide variety of soluble salts is typically found in soils. Two soils having the same resistivity may have significantly different corrosion characteristics, depending on the specific ions available. The major constituents which accelerate corrosion are chlorides, sulfates and the acidity (pH) of the soil. Calcium and magnesium tend to form insoluble oxide and bicarbonate precipitates, in basic environments, which can create a protective layer over the metal surface and reduce corrosion activity. Chloride ions tend to break down otherwise protective surface deposits, and can result in corrosion of buried metallic structures and reinforcing steel in concrete structures. Sulfates in soil can be highly aggressive to portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. High concentrations of bicarbonates tend to decrease soil resistivities. However, bicarbonates are not aggressive to buried steel and concrete. Although bicarbonates are not aggressive, lower resistivity environments can promote corrosion activity.

The following tables correlate the effect of chlorides on the rate of corrosion of steel, ductile iron or reinforcing steel in concrete, and the effect of sulfates on concrete:

Water Soluble Chloride Concentration (ppm)	Degree Of Corrosivity
Over 5,000	Severe
1,500 - 5,000	Considerable
500 - 1,500	Corrosive
Below 500	Threshold

Reference: Extrapolation from California Test Method 532, Method for Estimating the Time to Corrosion of Reinforced Concrete Substructures and V & A Consulting Engineers' experience

Water Soluble Sulfate Concentration (ppm)	Degree Of Corrosivity
Over 10,000	Very Severe
1,500 - 10,000	Severe
150 - 1,500	Moderate
0 - 150	Negligible

Reference: ACI-318, Building Code Requirements for Reinforced Concrete

Acidity, as indicated by the pH value, is another important factor of soil. The lower the pH (the more acidic the environment), the higher will be the corrosivity with respect to buried metallic and concrete structures. As pH increases above 7 (the neutral value), conditions become increasingly more alkaline and passive to buried steel structures. V & A has developed a table correlating the effect of pH on the rate of corrosion of buried steel or concrete structures. The data is derived from V & A's extensive experience and review of the literature, e.g., Romanoff M., Underground Corrosion, and Uhlig H., Corrosion and Corrosion Control.

pH	Degree Of Corrosivity
< 5.5	Severe
5.5-6.5	Moderate
6.5-7.5	Neutral
> 7.5	Negligible

RECOMMENDATIONS

Based upon the soil corrosivity data and review of the project requirements, corrosion monitoring and control will be required to prevent corrosion of metallic and reinforced structures. These recommendations are in accordance with Caltrans Bridge Design Specifications, Section 8.22. The quality of materials and workmanship should be inspected during construction by a corrosion engineer

Prestressed Concrete Piles

Buried reinforced concrete structures should be constructed of durable concrete such as described in ACI Standards 201.2R and 222R. These recommendations include but are not limited to the following:

- The water/cement ratio should not exceed 0.40.
- A concrete cover of a minimum of 2 inches should be applied over all steel reinforcement.
- Salt-free sand and potable water should be used.
- The concrete should be allowed to cure according to the manufacturer's recommendations.

- Mineral admixtures conforming to ASTM Designation C1240 and/or ASTM Designation C618 Type F and/or N should be used.

Buried Metallic Structures

Based on the low soil resistivity it is recommended that buried metallic piping systems, such as steel and ductile iron, be coated with a bonded dielectric or mortar coating. All non-welded joints should be bonded to provide electrical continuity. Cathodic protection should be installed to provide corrosion protection. Buried metallic piping should be electrically isolated from other structures, or where a change in piping or coating material occurs.

Buried Reinforced Concrete Structures

Buried reinforced concrete structures should be constructed of durable concrete such as described in ACI Standards 201.2R and 222R. These recommendations include but are not limited to the following:

- The water/cement ratio should not exceed 0.40.
- A concrete cover of a minimum of 3 inches should be applied over all steel reinforcement.
- Salt-free sand and potable water should be used.
- The concrete should be allowed to cure according to the manufacturer's recommendations.

MSE Retaining Walls

For permanent structures, the reinforcement elements shall be designed to have a minimum design life of 75 years. For the metallic reinforcement, galvanization loss will be:

- 15 μm/year/side for the first 2 years
- 4 μm/year/side for subsequent years

The loss of bare steel after the galvanized zinc is depleted will be 12 μm/year/side. These estimates are from the AASHTO Standard Specifications for Highway Bridges, Section 5.8.6.1. It is assumed that the soil backfill used is nonaggressive (i.e. Class 1 Backfill), meeting the following criteria:

- pH of 5 to 10
- Resistivity of not less than 3,000 ohm-cm

- Chlorides not greater than 100 ppm
- Sulfates not greater than 200 ppm.

The above mentioned thicknesses are not valid if the following conditions are present:

- The MSE wall is exposed to marine or other chloride rich environments
- The MSE wall will be exposed to stray currents such as from nearby underground power lines or adjacent railways
- The backfill material is aggressive

Please call if you have any questions or comments.

Sincerely,
V & A CONSULTING ENGINEERS
The Infrastructure Preservation People

Glenn H. Willson, PE
Project Manager